American Society of Civil Engineers

VOLUME 85 NO. HY5

MAY 1959

JOURNAL of the

Hydraulics Division

PROCEEDINGS OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

BASIC REQUIREMENTS FOR MANUSCRIPTS

This Journal represents an effort by the Society to deliver information to reader with the greatest possible speed. To this end the material herein none of the usual editing required in more formal publications.

Original papers and discussions of current papers should be submitted to Manager of Technical Publications, ASCE. Authors should indicate the techn division to which the paper should be referred. The final date on which a cussion should reach the Society is given as a footnote with each paper. Th who are planning to submit material will expedite the review and publica procedures by complying with the following basic requirements:

1. Titles should have a length not exceeding 50 characters and spaces.

2. A 50-word summary should accompany the paper.

3. The manuscript (a ribbon copy and two copies) should be double-sp on one side of 8½-in. by 11-in. paper. Papers that were originally prepared oral presentation must be rewritten into the third person before being submitt

4. The author's full name, Society membership grade, and footnote refere stating present employment should appear on the first page of the paper.

5. Mathematics are reproduced directly from the copy that is submitt Because of this, it is necessary that capital letters be drawn, in black ink, ⅛ high (with all other symbols and characters in the proportions dictated standard drafting practice) and that no line of mathematics be longer than 6½ Ribbon copies of typed equations may be used but they will be proportiona smaller on the printed version.

6. Tables should be typed (ribbon copies) on one side of 8½-in. by 11 paper with a 6½-in. by 10½-in. invisible frame. Small tables should be grou within this frame. Specific reference and explanation should be made in the for each table.

7. Illustrations should be drawn in black ink on one side of 8½-in. by 11 paper within an invisible frame that measures 6½-in. by 10½-in.; the capti should also be included within the frame. Because illustrations will be redu to 69% of the original size, the capital letters should be ⅛-in. high. Photogra should be submitted as glossy prints in a size that is less than 6½-in. by 10½ Explanations and descriptions should be made within the text for each illustrati

8. Papers should average about 12,000 words in length and should be longer than 18,000 words. As an approximation, each full page of typed te table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is c tained in the "Technical Publications Handbook" which can be obtained fr the Society.

This Journal is published monthly by the American Society of C Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michig Editorial and General Offices are at 33 West 39 Street, New York 18, New Yo $4.00 of a member's dues are applied as a subscription to this Journal. Seco class postage paid at Ann Arbor, Michigan.

Journal of the

HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

May, 1959

Papers

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

HYDROLOGY OF LAKE ONTARIO

F. I. Morton,[1] and H. B. Rosenberg[2]

SYNOPSIS

description of the physical and climatological features of the Lake rio basin is presented. The effects of climate and topography on the ly of water into Lake Ontario are discussed. Short and long term records ows and supplies of Lake Ontario are related to the factors causing varia- in the water supplies. Detailed descriptions of the source of supply are in relation to seasonal and areal distribution of precipitation, tempera- and runoff. Flow records of many of the large tributaries of Lake rio are analysed and the possibility of long and short-term forecasting is ssed.

INTRODUCTION

ne International St. Lawrence River Board of Control is currently de- ing plans for regulating the levels and outflows of Lake Ontario in con- ion with the St. Lawrence Seaway and Power projects. The technical es are being performed jointly by various governmental agencies in the d States and Canada. The Water Resources Branch of the Department of ern Affairs and National Resources is one of the Canadian agencies as- ted with this work. The problem is international in all its ramifications the International Boundary traverses not only Lake Ontario and the St. ence River to the vicinity of Cornwall, Ontario, but also the upper Great s and their connecting channels with the exception of Lake Michigan.

Discussion open until October 1, 1959. To extend the closing date one month, a itten request must be filed with the Executive Secretary, ASCE. Paper 2017 is rt of the copyrighted Journal of the Hydraulics Division, Proceedings of the nerican Society of Civil Engineers, Vol. 85, No. HY 5, May, 1959.

dr. Engr., Water Resources Branch, Dept. of Northern Affairs and tional Resources, Ottawa, Ontario, Canada.

dr. Engr., Water Resources Branch, Dept. of Northern Affairs and tional Resources, Ottawa, Ontario, Canada.

In the preliminary regulation studies which have already been made, pre-conceived rules have been developed and tested under historical water suppl conditions. The water supplies utilized in testing the plans have been derive from historical outflow and water level changes and results beneficial to all interests have been achieved without the necessity for a detailed study of the sources of supply and the other hydrologic factors. If further refinements are to be made, operating experience as well as a more detailed knowledge c hydrologic data will be required.

The question may be asked as to what may be gained from a better understanding of the various factors contributing to the water supply to Lake Ontario. Firstly, the knowledge gained from such a study will contribute to the confidence of those responsible for the operation of the works when the actual water supplies to be regulated will assuredly be different from those which have occurred in the past which have been the basis for the development of the regulation plan. The hydrology of Lake Ontario is very complex so that a thorough understanding of it will be necessary for efficient, flexible operation of the control works. Secondly, such studies may enable the development of long-term and short-term forecasts of water supplies. This would contribute to the improved use of the available storage on Lake Ontari Thirdly, this knowledge can make possible an assessment of the possible advantages to Lake Ontario of the development of regulation facilities on the upper Great Lakes.

This study was prepared with these considerations in mind. Its principal purpose was to investigate in a general way all the factors influencing the inflows to Lake Ontario and to set them forth in a concise form. The effects o precipitation, temperature, lake areas and the physiography of the drainage basin on the character of the inflow are analysed. From the better understanding of inflow characteristics gained from this analysis an assessment is made of the probable effects on Lake Ontario of the regulation of the upper Great Lakes and the possibility of long and short-term forecasts of water supplies.

Basin Characteristics

General

The Great Lakes constitute one of the finest natural regulatory systems in the world. At the outlet of Lake Ontario the total drainage area of the system is about 300,000 square miles of which approximately one-third is water surface. The excess water from the upper Great Lakes, i.e. Superior, Michigan-Huron, St. Clair and Erie, passes through Lake Ontario and thence to the St. Lawrence River and the sea. This water which is transmitted to Lake Ontario from the upper Great Lakes is of great importance to an understanding of the hydrology of Lake Ontario since it comprises approximately 85% of the total water supply to the lake. Figure 1 is a map of the drainage basin of the Great Lakes. Figure 2 presents a summary of the pertinent physical data.

Upper Great Lakes

The characteristics of the water supply to the upper Great Lakes are subject to the divergencies in climate, topography and geology that can occur in

FIGURE-1

FIGURE-2

PHYSICAL DATA PERTAINING TO THE GREAT LAKES

Lake	Drainage Area		Storage Capacity Per Foot of Stage of Lake c.f.s. for one Month	Average Elevation m.s.l.	Range of Monthly Mean Stage	Outlet River	Mean Outflow	
	Land Area Square Miles	Water Surface Area Square Miles					c.f.s.	Inches on Total Drainage Basin
Superior	49,000	31,800	337,000	602.3	4.1	St. Marys	75,000	12.6
Michigan Huron	46,600 49,400	22,400 23,000	481,000	580.6	6.2	St. Clair Lake St. Clair Detroit	189,000	11.6
Erie	29,400	9,900	105,000	572.4	5.4	Niagara	205,000	10.6
Ontario	27,100	7,500	80,000	246.0	6.6	St. Lawrence	241,000	11.1
	201,500	94,600						

an area of 260,000 square miles extending 750 miles from east to west, an 650 miles from north to south.

The region contributing to the water supply of Lake Superior and the northern shore of Lake Huron is generally rough, wooded country typical o the Precambrian Shield. To the south and east of Lake Erie lies the Allegl plateau which is part of the Appalachian formation. Between these two formations lie the Great Lakes - St. Lawrence Lowlands which comprise the greater part of the area draining into the upper Great Lakes system. The glacial periods had a great effect on the entire region, grinding down and denuding the Precambrian Shield, flooding the low areas and depositing silt and sands, while leaving morainic hills, till plains, drumlins and eskers or the more elevated parts and generally disarranging old drainage patterns.

The upper Great Lakes region lies in the path of the prevailing westerly winds and is particularly subject to winter storms. In the lee of the lakes areas of more moderate temperatures and extremely high snowfalls. Generally, the temperature increases from north to south and the precipitation increases from northwest to southeast. The northern part of the drainage basin has a mean annual temperature of 31°F. with an average length of the frost-free period of only 75 days. Near the western end of Lake Erie the mean annual temperature exceeds 48°F. and the average length of frost-fre period exceeds 175 days. The average annual rainfall varies from 25 inche west of Lake Superior, to over 40 inches to the south and east of Lake Erie

Although the inflows to the upper Great Lakes and the evaporation losse vary widely from year to year, from season to season and from lake to lak the diversified runoff characteristics and the great expanses of lake surfac contribute to a relatively uniform inflow to Lake Ontario through the Niaga River. The monthly mean outflows from Lake Erie have varied from a hig of 254,000 cubic feet per second to a low of 121,000 cubic feet per second.

For comparative purposes the outflows from the various lakes which ar shown on Figure 2 and the outflows from Lake Erie just mentioned are tho that have been derived from actual records. Certain adjustments would be necessary to make these flows consonant with present conditions of diversi into and out of the Great Lakes System and any further references to flows and levels will include these adjustments. A net diversion into the Great Lakes system of 1900 cubic feet per second is presently in effect. This includes the 3100 cubic feet per second diverted out of Lake Michigan at Chi and the 5000 cubic feet per second diverted into Lake Superior from the Albany River system.

Lake Ontario Drainage Basin

General

The local drainage basin of Lake Ontario has an area of 34,630 square miles including 7,540 square miles of lake surface or over 20 per cent of t local drainage basin. Of the drainage area, 18,710 square miles are in the United States with the remaining 15,920 square miles in Canada. From we to east the greatest length of the drainage basin is 280 miles and the great width from north to south is 240 miles. The lake has an average depth of a proximately 250 feet with a maximum recorded depth of almost 800 feet. T average water level has been about elevation 246 and the maximum range o monthly stages has been about 6.6 feet.

siography

he geological features which dominate the local drainage area of Lake rio are the Great Lakes - St. Lawrence Lowlands, the Precambrian ld, the Adirondack Mountains and the Allegheny plateau. These features highlighted on the contour map of the local drainage basin presented on ire 7-A.

he Great Lakes - St. Lawrence Lowlands follow the shoreline of Lake rio, striking inland for various distances. The lower lands close to the became covered with silts and sand. At the edge of this lower area are e-cut cliffs, beaches, bars and deltas. To the south and west of the lake Niagara escarpment forms another boundary to this lower-lying area. In higher areas the relief was formed by the shoving and depositing action e glaciers which left behind the morainic hills, drumlins, eskers and till ns. The glaciers also blocked off the established southward drainage of Finger Lakes in New York State causing them to flow northward into Lake rio. This area includes the headwaters of the small tributaries to the tern portion of Lake Ontario.

The Allegheny plateau forms the northern edge of the Appalachian forma- . It extends across the southern end of the drainage basin in an east-west ction from Syracuse to Buffalo and is deeply indented by the Finger Lakes. plateau slopes upward to the south and ranges in elevation from 1,000 to 0 feet above sea level. The terrain is very irregular and hilly and has largely cleared of its original forest growth. The highest point that is ieved in this formation in the drainage basin is 2,550 feet above sea level. headwaters of the Genessee River and the southern and western tribu- es of the Oswego River lie within this area.

The northern part of the drainage basin is located within the Precambrian ion. Rock knobs or hills with shallow soil depths predominate. Little of area is cultivated and it is still largely covered with forest. In the ex- ne north of the drainage basin elevations of over 1,500 feet above sea l are reached but the general elevation decreases to the south and east to Thousand Islands Section of the St. Lawrence River. This region includes headwaters of the Trent, Moira and Napanee Rivers.

The Adirondack plateau is an elevated extension of the Precambrian Shield th of the St. Lawrence River and east of Lake Ontario. The plateau is well ded with very rough topography. The highest hill in the drainage basin ches an elevation of approximately 3,700 feet although elevations of over 0 feet are reached farther east. The area includes the headwaters of the vegatchie and Black Rivers and the northeastern tributaries of the Oswego er. The valley of the Black River indents the plateau in a striking fashion.

mate

The Lake Ontario drainage basin lies along the track of many of the low ssure areas which sweep across the northern part of North America from t to east. This results in stormy changeable weather with wide variations emperature but remarkably uniform annual precipitation. For the drain- basin as a whole the annual precipitation since 1870 has varied from 25 to inches with a mean of 33 inches. The prevailing westerly winds combined the topography play a major part in the areal distribution of rainfall. The rage annual precipitation varies from 27 inches in the area below the gara escarpment to over 50 inches in the Adirondacks to the east of the e. The snowfall is even more variable with the prevailing winds picking up

a large quantity of moisture as they pass over the open lake and depositing to the lee of the lake. The average snowfall varies from 40 inches at the w of the lake at St. Catherines to 175 inches on the eastern end, south of Wate town.

The temperatures in the drainage basin are variable with the area below the Niagara escarpment having the highest temperatures, and the northern part of the drainage basin and the higher elevations of the Adirondacks havi the lowest temperatures. The average annual temperature is about 45°F wi extremes of temperatures of 105°F and -40°F having been recorded.

Water Supply to Lake Ontario

General

An analysis of the water supply conditions for Lake Ontario will be developed along two different but complimentary approaches. An analysis wil first be made of the long term records and then supplemented by considerat of shorter term records with more comprehensive coverage. Because of their importance to navigation, continuous water level records for the Grea Lakes and the St. Lawrence River have been kept since 1860. It has been possible, therefore, to construct records of the water supplies and outflows of the lakes for a period of almost one hundred years. Records of tributary flows and detailed climatological records, however, have been available for a much shorter period of time.

Net Total Water Supply to Lake Ontario

The net total water supply to Lake Ontario is defined as the inflow to the lake which is available for outflow. It is equal to the inflow from the tributary area plus the precipitation on the lake surface less the evaporation fro the lake surface. The records of monthly net total water supplies have bee based on the monthly mean discharge of the St. Lawrence River at Iroquois, Ontario, with the water levels at the beginning and end of the month measur at selected gauges around the lake shore. A change of one foot in the elevation of Lake Ontario in one month indicates that the difference between the net total water supply and the outflow has been 80,000 cubic feet per second The recorded net total water supplies have been adjusted to make them consonant with a continuous diversion into the upper Great Lakes of 1900 cubic feet per second.

The net total water supply to Lake Ontario may be broken down into two principal components, the outflows from Lake Erie and the net local water supply. The next total water supply has averaged 246,000 cubic feet per second with 210,000 cubic feet per second contributed from Lake Erie and the balance from the local drainage basin. The average monthly distributio of these two components, as well as the total is shown on Figure 3-A. In general the net total water supply reaches a maximum in the month of April and a minimum in the month of October. Most of the variation in the avera monthly distribution is due to effect of the net local water supply. Figure 3 illustrates the seasonal distribution of the extremes of the net total water supply. The highest and the lowest lines are the maximum and minimum fo each month regardless of the year in which they occurred. Generally, the extremes vary in the same manner as the averages. The monthly mean net

vater supplies have varied from 416,000 cubic feet per second to 0C cubic feet per second. The crosshatched area in Figure 3 represents ;e of one standard deviation on either side of the mean. The standard ion varies little throughout the year.

nth-to-month variations in the net total water supplies are generally small in the late summer and early fall but can be very large during the and spring. An extreme value occurred in 1936 when the variation in onthly means from February to March was 178,000 cubic feet per d. An example of extreme variation for three months occurred in 1913 the monthly mean net total water supplies for January, February and were 324,000, 220,000 and 326,000 cubic feet per second.

e variation in the mean annual net total water supplies is shown on a ency distribution on Figure 4. The extreme variation for the yearly has been 93,000 cubic feet per second or 38 per cent of the average.

w from Lake Erie

e greater part of the excess water from the upper Great Lakes passes h the Niagara River into Lake Ontario. To obtain the total outflows Lake Erie the quantities of water diverted for power and navigation have dded to the Niagara River flows. These total outflows have also been ed for a continuous net diversion into the upper Great Lakes of 1900 feet per second.

e flow in the Niagara River is essentially dependent on the water levels e Erie. There are three categories of variation in the outflow from Erie, i.e. transient, seasonal and long term.

e transient variations are caused by winds and seiches (oscillation of aters due to changes in barometric pressure) which may affect the level of Lake Erie at the head of Niagara River by more than ten feet, ice jams forming at the outlet of the lake at the head of the Niagara A large variation in flow resulting from a recent storm is shown on rly hydrograph in Figure 5.

sonal variations in the outflows from Lake Erie are small due to the ating effect of the storage on Lake Erie and the other upper Great Generally the outflow reaches its peak during the month of June and est value during the months of January or February. The monthly of the average outflows is shown in Figure 3-A.

diversity of hydrologic factors affecting the water supplies to the up- eat Lakes and the retarding effect of the great volume of storage avail- esult in long-term variations in the outflows from Lake Erie. Short- ariations in water supply are absorbed by the storage with little effect outflows of Lake Erie but long-term variations gradually change the s so that the seasonal and transient variations are superimposed upon r or lower base.

example of the seasonal and long-term variation of the outflows from rie from 1926 to 1935 inclusive is shown in Figure 6. Also shown is graph of average outflows and a plot of the accumulated deviations verage of the precipitation on the drainage basin of the upper Great The flow gradually built up from much below average in 1926 to near ximum recorded in 1929 and then decreased to the minimum recorded . These long-term variations were obviously in response to long term ons in the precipitation on the drainage basin. The figure very clearly

FIGURE

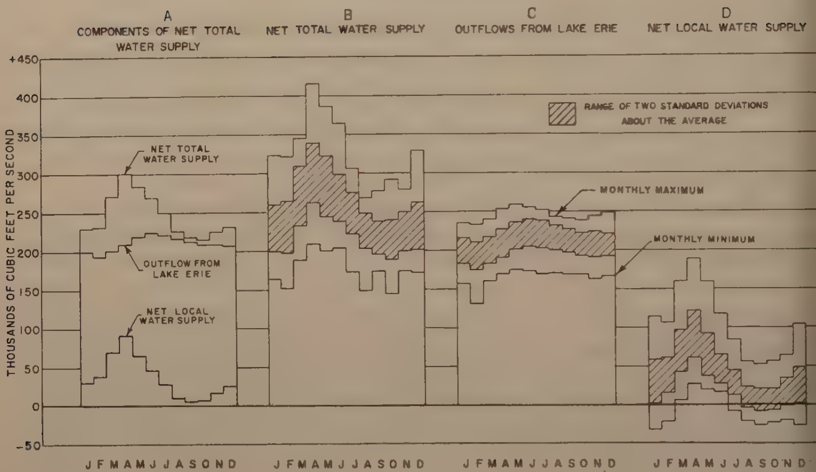


FIGURE-4

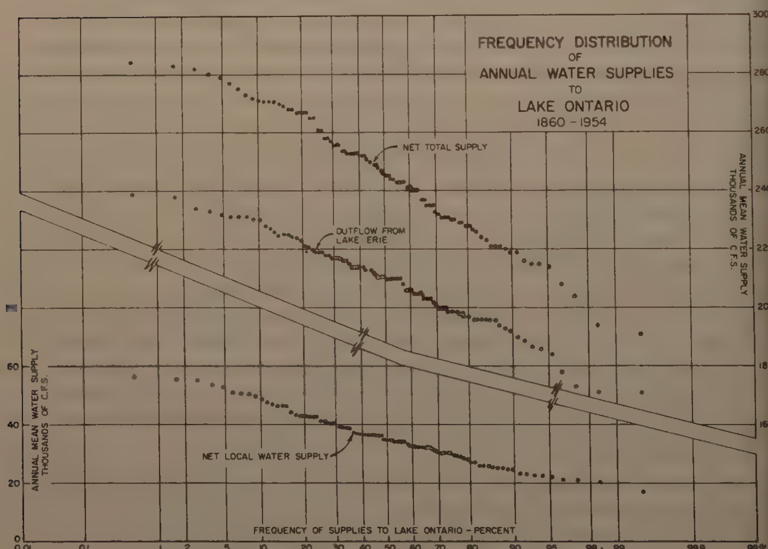


FIGURE-5

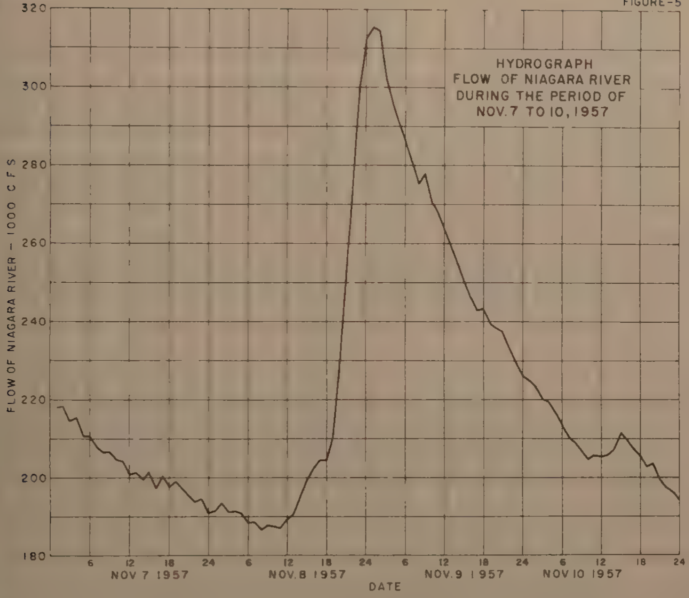
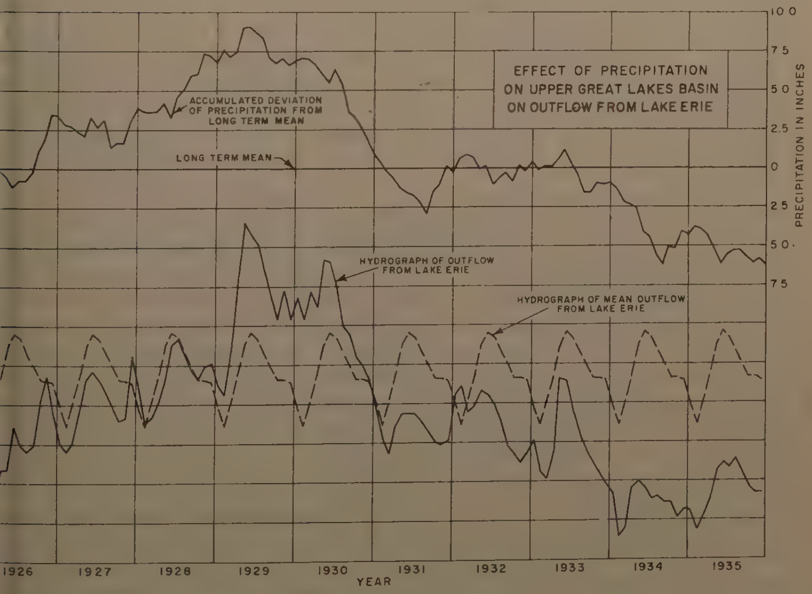


FIGURE-6



shows the seasonal variations which were superimposed on a base that was much above or below average. Although the precipitation for 1932 and the early part of 1933 was average, the deficiency of water supply to the upper Great Lakes which occurred in the extremely dry period in 1931 had so de- pleted the storage on the lakes that the outflow from Lake Erie continued t be much below average.

The seasonal distribution of extreme high and low monthly mean outflow from Lake Erie, regardless of when they occurred, is shown in Figure 3-C The trend of these extremes follows the trend of average seasonal variatio quite closely. The monthly mean outflows have varied from a maximum of 258,000 cubic feet per second to a minimum of 131,000 cubic feet per seco or a range of about 60% relative to the mean. The minimum occurred in February 1936 and a study of available records indicates that it was cause in part by a combination of easterly winds and an ice jam at the head of the Niagara River.

A range of one standard deviation on either side of the average is also shown. The standard deviation varies very little during the year.

The variations in the annual mean outflows from Lake Erie are shown o Figure 4 in the form of a frequency distribution. The maximum variation the annual means has been 68,000 cubic feet per second or about 32% of the mean.

Net Local Water Supply

The net local water supply to Lake Ontario is the difference between the net total water supply and the outflow from Lake Erie. Since it is based on the difference between two large values it is possible that the percentage error in its determination may be high. The net local water supply is com posed of the land runoff from the local drainage basin of Lake Ontario plus the precipitation on the lake surface less the evaporation from the lake sur face. The seasonal distribution of the average net local water supply is shown on Figure 3-A. The maximum monthly average was 90,000 cubic fe per second in April and the minimum was 6,000 cubic feet per second in September.

The seasonal distribution of the maximum and minimum net local water supply for each month is illustrated on Figure 3-D. It may be noted that d ing the late summer, fall, and early winter, the minimum net local water supplies are negative quantities. These negative quantities result when the evaporation from the lake surface exceeds the sum of the precipitation on the lake surface and the land runoff. The net local water supply varies wi with a maximum monthly mean of 188,000 cubic feet per second and a mini mum monthly mean of -32,000 cubic feet per second or a range of about 61 of the mean. The seasonal distribution of the extremes bears a close rela tion to the seasonal distribution of the averages. A range of one standard deviation on either side of the mean for the net local water supply is also shown. The standard deviation varies from a maximum in April to a mini- mum in August. The standard deviations for the net local water supply ave age 70 per cent of the standard deviations for the net total water supply.

The net local water supply varies widely from month to month, notably the winter and spring seasons. The maximum variation occurred in 1936 the variation from February to March was 138,000 cubic feet per second. During the year 1913 the mean monthly net local water supplies varied fro

000 c.f.s. in January, to 5,000 c.f.s. in February, to 110,000 cubic feet second in March.

igure 4 shows the variation in the annual mean net local water supplies frequency distribution. The maximum variation of the annual means has 39,000 c.f.s. or 109% of the average net local water supply. This variais 62% of the maximum variation in the annual outflows from Lake Erie. slope of the frequency distribution for the net total water supply is greater the slopes for either the outflows from Lake Erie or the net local water ly which indicates that there is a tendency for both components to be high w together. However, the extreme values of both components have not cided.

more thorough understanding of the variations in the net local water ly requires a more detailed study of the land runoff in the local drainage 1 and the precipitation-evaporation balance on the lake surface.

l Runoff

and runoff is the inflow to Lake Ontario from the local drainage area exve of the lake surface. This area is 27,090 square miles. The major taries are the Oswegatchie, Black, Oswego, Genessee, Trent and Moira rs which contribute more than half of the net local water supply to Lake rio and form the basis of the analysis of land runoff in this study.

he Oswegatchie and Black Rivers drain a large part of the Adirondack au within the Lake Ontario drainage basin. The flows of both tributaries ffected somewhat by the operation of storage reservoirs in the headrs. The Oswego River is the largest tributary of Lake Ontario exclusive e Niagara River. It drains the southeastern part of the drainage basin headwaters in both the Adirondack and Allegheny plateaus and serves as tlet for the Finger Lakes. Its flow is regulated appreciably in conjunc-vith the operation of the New York State Barge Canal. The Genessee rises in the Allegheny plateau of Pennsylvania and drains the southal part of the local drainage basin. Its flows are regulated slightly by eam storage reservoirs. The Trent River rises in the extreme northern of the drainage basin in the Precambrian Shield and drains the northal portion of the local drainage basin. It has numerous lakes along its e and the flows are regulated for power and for operation of the Trent system. The Moira River has its headwaters in the Precambrian d to the east of the Trent River and flows through a region with very soil cover. The flows are not regulated. The period of record for the stream gauging stations on these rivers varies from 26 to 45 years.

ne of the major tributaries arise in the Great Lakes - St. Lawrence nds to the west of the lake or in the northeastern part of the drainage Two smaller rivers, the Credit and the Napanee, are considered to presentative of these areas. The Credit River rises above the Niagara rpment and flows into Lake Ontario west of Toronto. The Napanee drainasin is similar to that of the Moira. The flows of the Credit and Napanee s are substantially unregulated. The periods of record at the downn gauging stations are 10 years for the Credit River and 21 years for apanee River.

e density of stream measurement stations on the local drainage basin is ate for the determination of the areal distribution of runoff. Figure 7-D ap of the drainage basin showing the distribution of mean annual runoff es. The area of maximum runoff is east of Lake Ontario in the

headwaters of the Black and Oneida Rivers. Here the average runoff is as high as 35 inches per year. The area of lowest runoff follows the shoreline of the western half of the lake where the average runoff is as low as 10 inch Generally the runoff increases with distance from the lake. The reason for this distribution is apparent from inspection of Figure 7-B showing isohyet of mean annual precipitation. From a comparison of Figures 7-B and 7-D, may be deduced that the annual average water loss is generally about 20 inc although it is somewhat less to the east of the lake. It may be seen that the areas of greatest precipitation and runoff are coincident with the highest la In addition, the prevailing westerly winds, after crossing Lake Ontario, are affected orographically by the Adirondacks. This is especially true in the winter time where the winds sweeping over the open water pick up large quantities of moisture and drop it in the form of snow in the lee of the lake. The distribution of the mean annual snowfall is shown in Figure 7-C.

The seasonal distribution of average precipitation on the drainage area i shown on Figures 8A, 8B, 8C and 8D. The areal distribution of precipitatio during any one season generally follows the same pattern as that for the ye A remarkable feature illustrated by these figures is that the precipitation does not vary widely from season to season. Figures 9A, 9B, 9C and 9D sh the areal variation of average temperatures for the months, January, April July and October respectively. The decrease in temperature with both lati- tude and altitude is apparent. The range of areal variation of temperature greatest in the winter. The seasonal and areal variations in runoff are sho on Figures 10A, 10B, 10C and 10D. During the winter season the effect of areal variations of temperature on the runoff is very marked. In the northe sections of the basin and in the Adirondacks, where the temperatures are lowest, the precipitation is stored in the form of snow so that the runoff is much less than the precipitation. In the lower and more southern part of th drainage basin a combination of higher temperatures, small evaporation an transpiration losses contribute to a runoff that is almost as great as the pr cipitation. During the spring months the situation is reversed. In the area where the precipitation was stored in the form of snow the runoff is either greater or slightly less than, the precipitation, whereas in the other areas is considerably less. During the summers the large evaporation and trans piration losses result in a generally low runoff. In the fall months the wate losses decrease so that the runoff becomes generally higher. The areal va ations of average water losses during any season do not seem to vary signi cantly. Thus, although the precipitation does not vary much from season to season the effects of temperature and topography cause large seasonal fluc tuations in runoff.

Extreme high and low monthly mean precipitation and temperatures at f stations are shown on Figure 11. A range of one standard deviation on eith side of the average precipitation is also shown. The stations at McKeever, Alfred, N.Y., and St. Catharines, Apsley, and Kingston, Ontario, are con- sidered to be representative of the Adirondack plateau, the Allegheny plate the western end of Lake Ontario, the Precambrian Shield and the eastern e of Lake Ontario respectively.

It can be seen that, with the exception of Alfred, the average precipitatio does not vary much from month to month. The higher summer average pre cipitation at Alfred indicates a somewhat different climatic condition at the extreme southern end of the local drainage basin. The greatest variations precipitation generally appear during late summer or autumn, possibly due

FIGURE 7-A

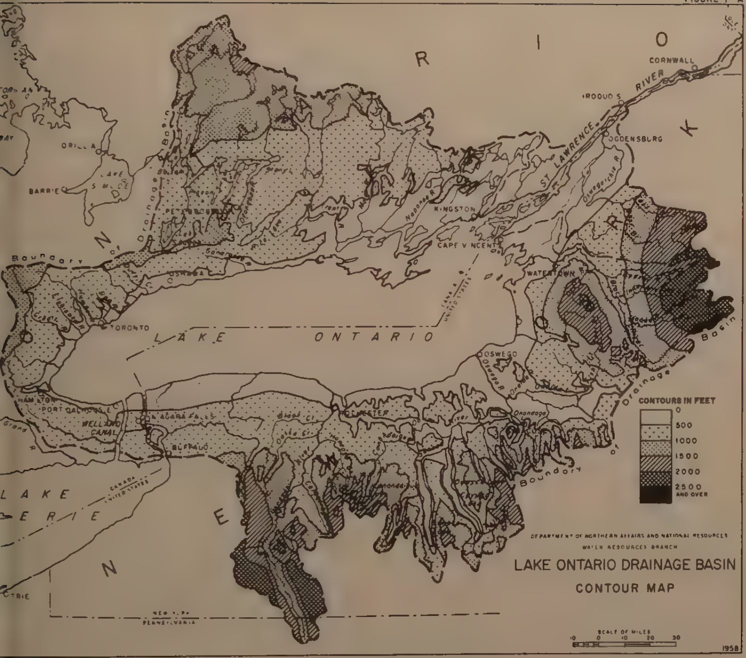


FIGURE 7-B

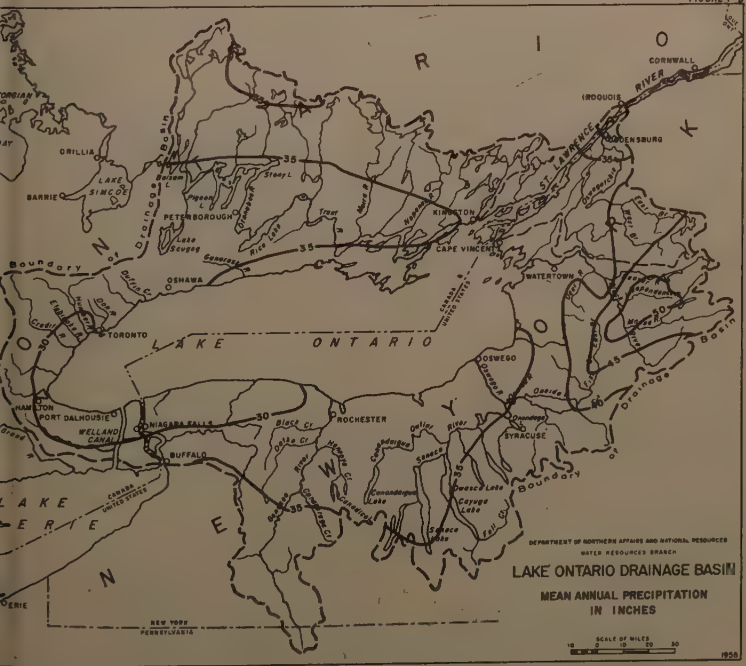


FIGURE 7-C

FIGURE 7-D

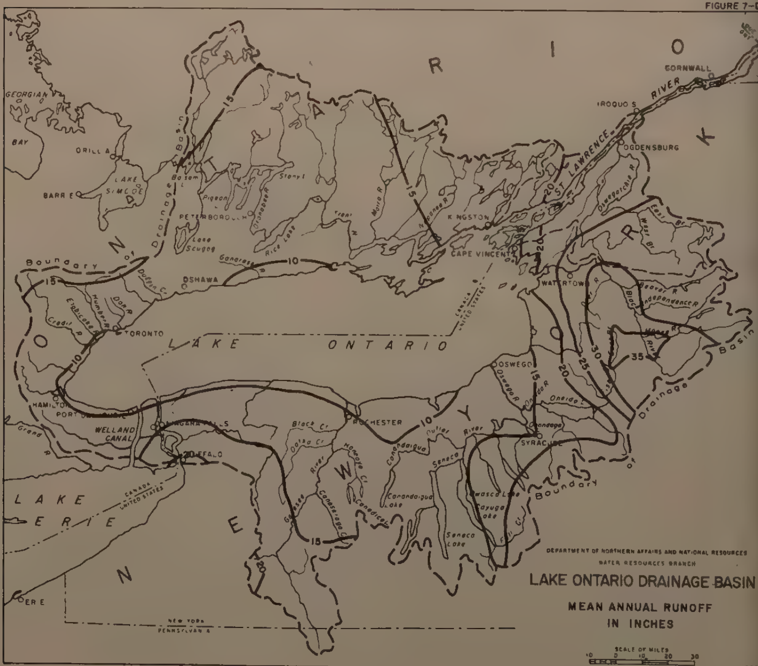
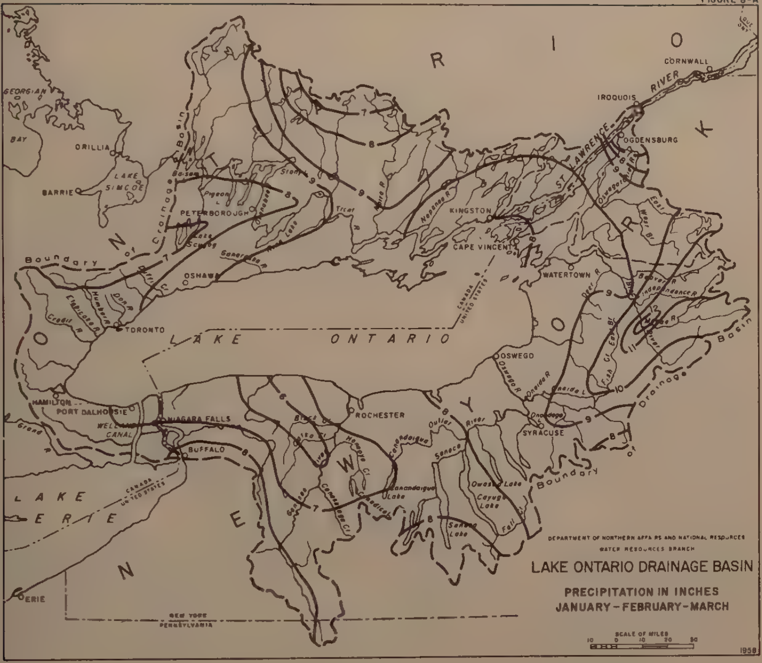


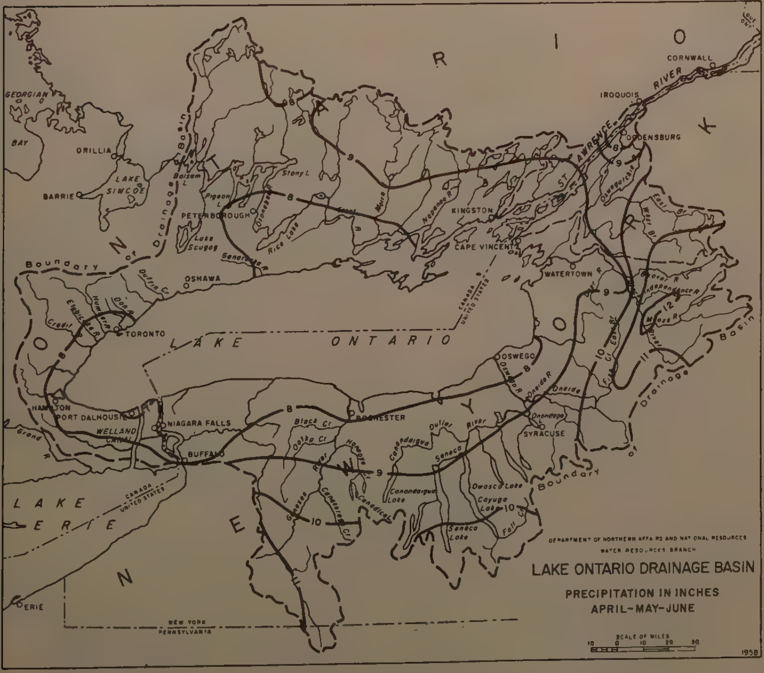
FIGURE 8-A



LAKE ONTARIO DRAINAGE BASIN

PRECIPITATION IN INCHES
JANUARY-FEBRUARY-MARCH

FIGURE 8-B



LAKE ONTARIO DRAINAGE BASIN

PRECIPITATION IN INCHES
APRIL-MAY-JUNE

FIGURE 8-C

FIGURE 8-D

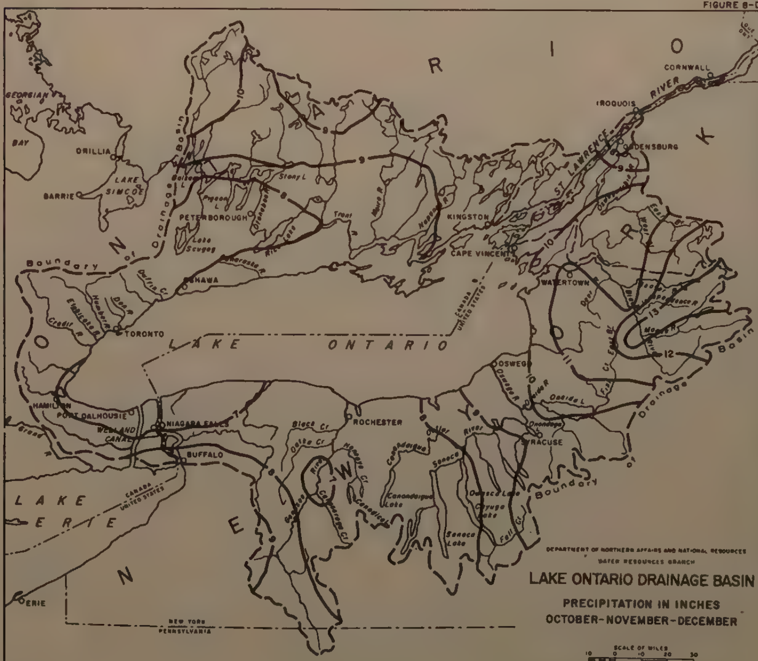
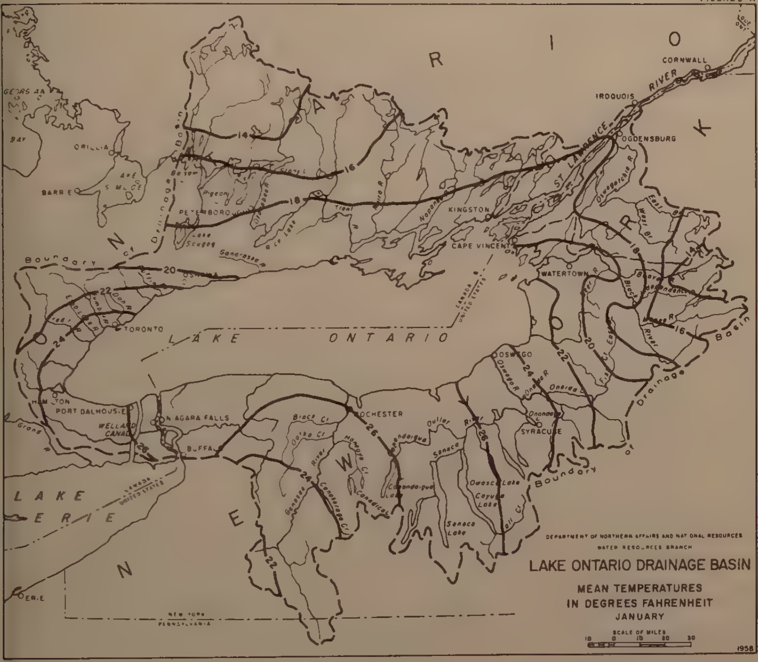
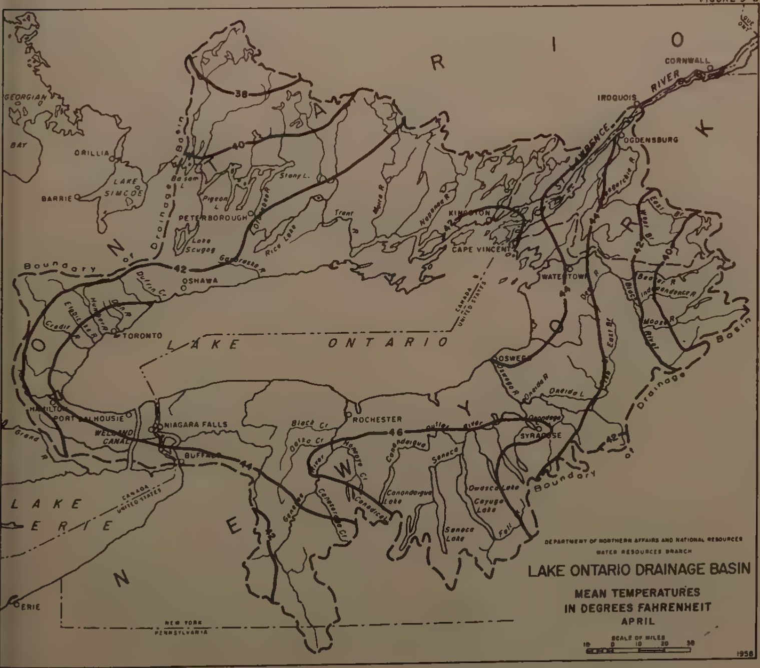


FIGURE 9-A



DEPARTMENT OF NORTHERN AFFAIRS AND NATIONAL RESOURCES
WATER RESOURCES BRANCH
LAKE ONTARIO DRAINAGE BASIN
MEAN TEMPERATURES
IN DEGREES FAHRENHEIT
JANUARY
SCALE OF MILES
1958

FIGURE 9-B



DEPARTMENT OF NORTHERN AFFAIRS AND NATIONAL RESOURCES
WATER RESOURCES BRANCH
LAKE ONTARIO DRAINAGE BASIN
MEAN TEMPERATURES
IN DEGREES FAHRENHEIT
APRIL
SCALE OF MILES
1958

FIGURE 9-C

FIGURE 9-D

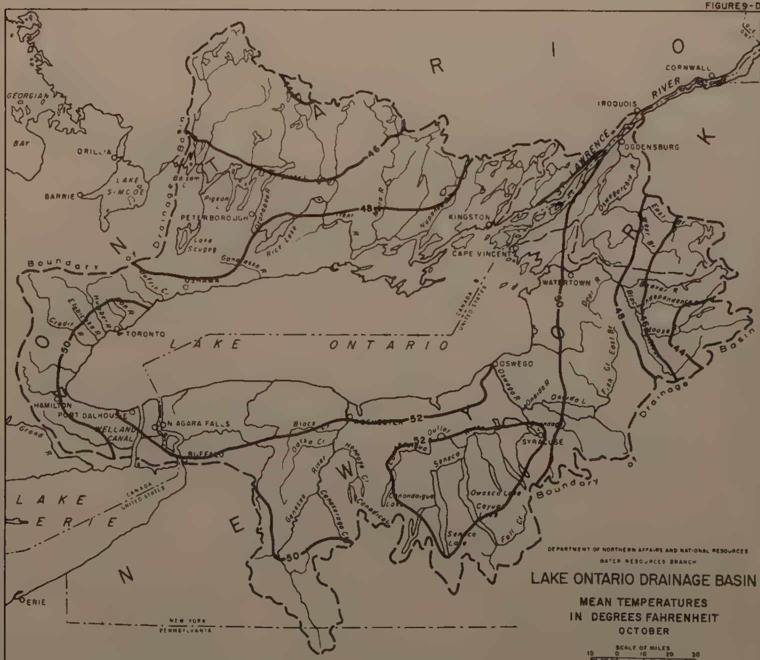


FIGURE 10-A

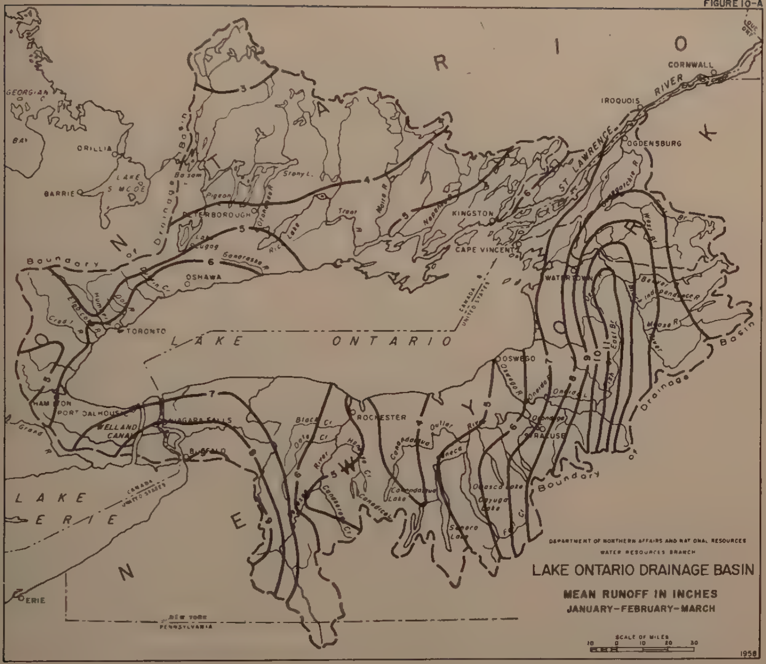


FIGURE 10-B

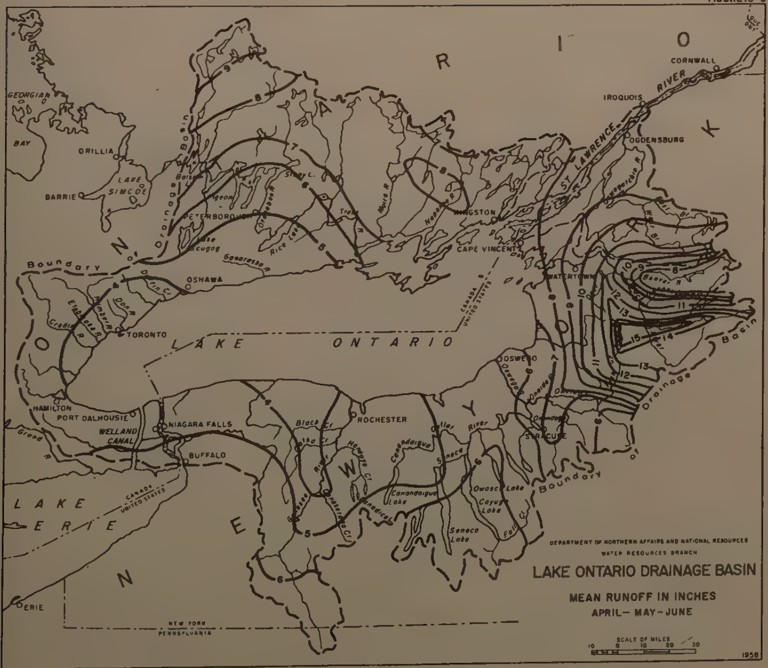


FIGURE 10-C

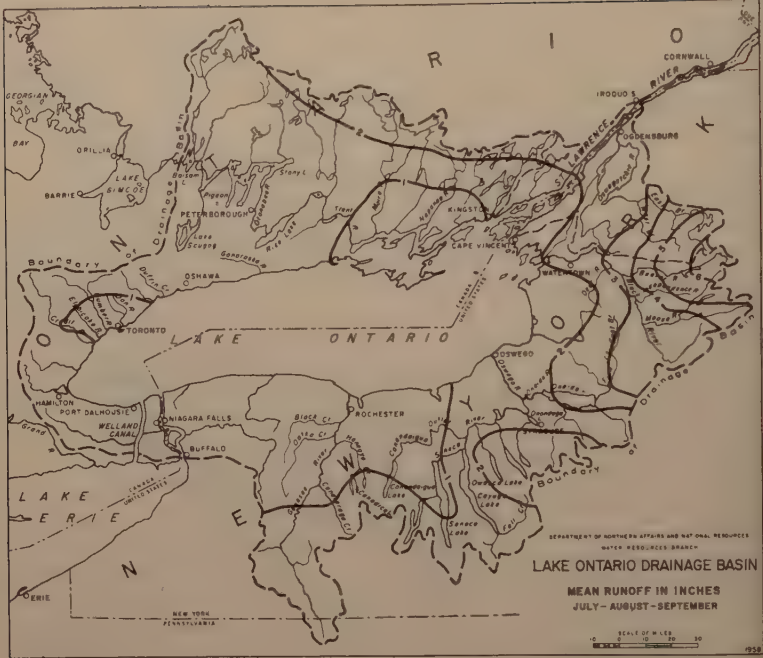
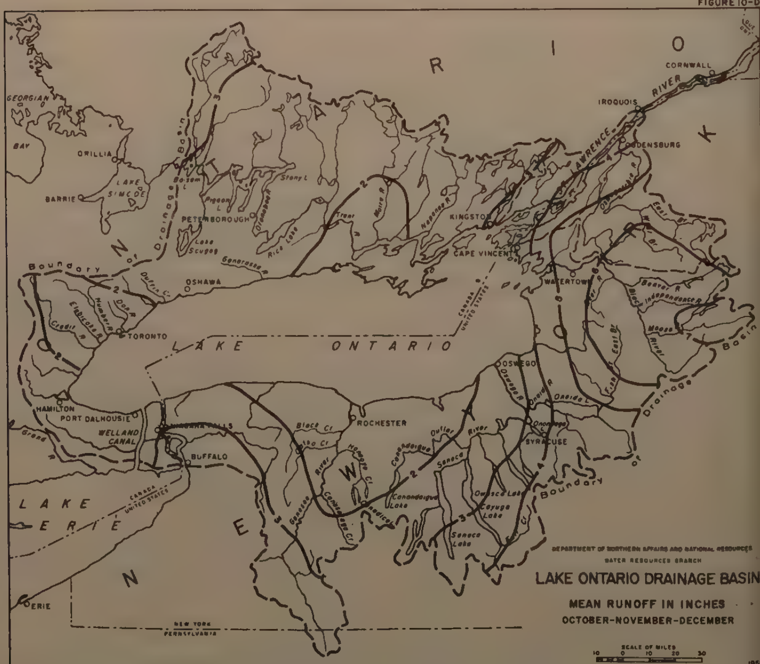


FIGURE 10-D



nderstorm activity or the occasional tropical storm which may cross the in. Although only the maximum and minimum monthly mean temperatures shown, the degree of variation of these quantities is an indication of the iation that can take place within the month. The winter period exhibits the est variations in mean temperatures. The minimum and maximum month- emperatures for all the stations follow the same trend. The similarity in nperature variation between two widely separated stations i.e. McKeever Apsley may be noted. Apparently the effect of increased elevation at Keever balances the effect of the northerly location of Apsley.

For a more detailed study of the seasonal distribution and variation of the d runoff it is necessary to examine more closely the behaviour of the in- idual tributaries. On Figure 12 is presented a hydrograph of the mean nthly flow in inches for various tributaries and for the net local water ply. Also shown is the percentage of the mean annual flow for each month. e much higher unit runoff from the Adirondack plateau into the Oswegatchie Black Rivers is immediately obvious. The lack of a deep soil cover and rage reservoirs in the Moira and Napanee drainage basins has a marked ect on the distribution of runoff. Much higher percentages of the annual v occur in the spring and much lower percentages in the late summer than h the other tributaries. Moreover, on these tributaries there is a smaller centage of the annual runoff during the winter which would indicate a ater accumulation of snowcover. This would tend to increase the spring off. Throughout the greater part of the local drainage basin of Lake ario the peak flow generally occurs in April although on the Genessee and edit Rivers in the western part of the basin the spring season seems to be re advanced. The Trent and Oswego Rivers have the most uniform dis- oution of average monthly flows because of their large size and of the ef- ts of regulation. With the exception of the Genessee and Credit Rivers the rage flow for January for all the tributaries is higher than for either cember or February. This would give some credence to the local tradi- n of a "January thaw." The runoff pattern for all the tributaries is, in te of these differences, generally similar to that for the net local water ply. The difference can generally be attributed to the effect of cipitation-evaporation balance on the lake. The maximum and minimum vs for each month are compared to the mean flows for each month on ure 13. The monthly distribution of the standard deviation is also shown. e figure illustrates that the range of variation is smallest in the summer nths and greatest in the winter and spring. The reason for the small ge in variation of runoff is that the evaporation and transpiration losses rease with the availability of water. The decrease in the effect of these ses in the fall allows a much greater range of runoff. The effect of large nperature variations on the snow that has fallen and the small water los- tend to make the variability of runoff during the winter extremely high. e variability during the spring period results from the variation of the w cover at the end of winter as well as the spring precipitation. Generally standard deviation follows the same pattern as average monthly runoff.

It may be concluded from the foregoing that the variability of runoff is de- dent not only on the variation of precipitation but also on the variation of nperature. A specific example in which the precipitation and temperature related to runoff is shown on Figure 14. Deviations in precipitation and nperature at Belleville for the period 1941 to 1950 are compared graphical- vith the deviations in flow of the Moira River. Belleville is at the mouth of

FIGURE-11

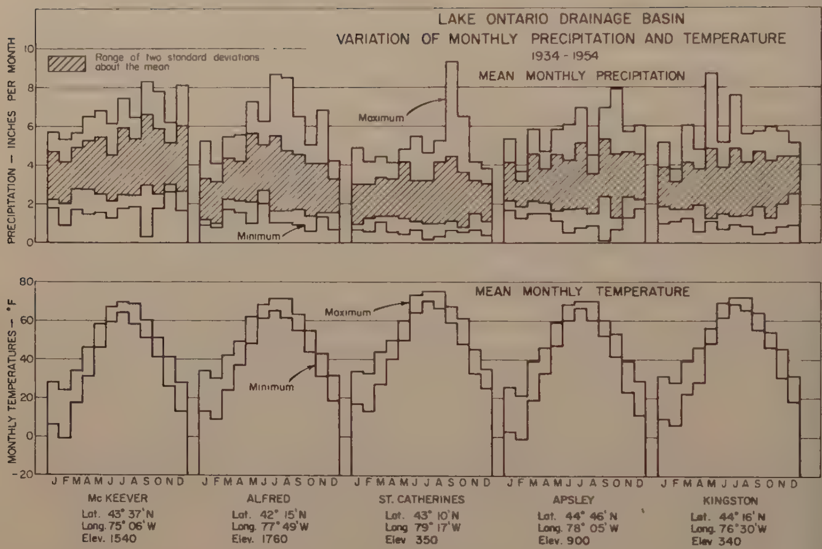
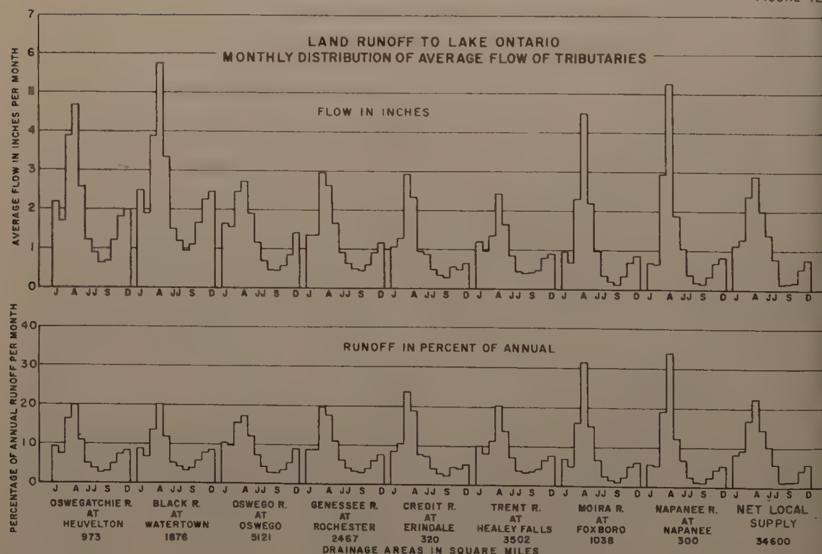
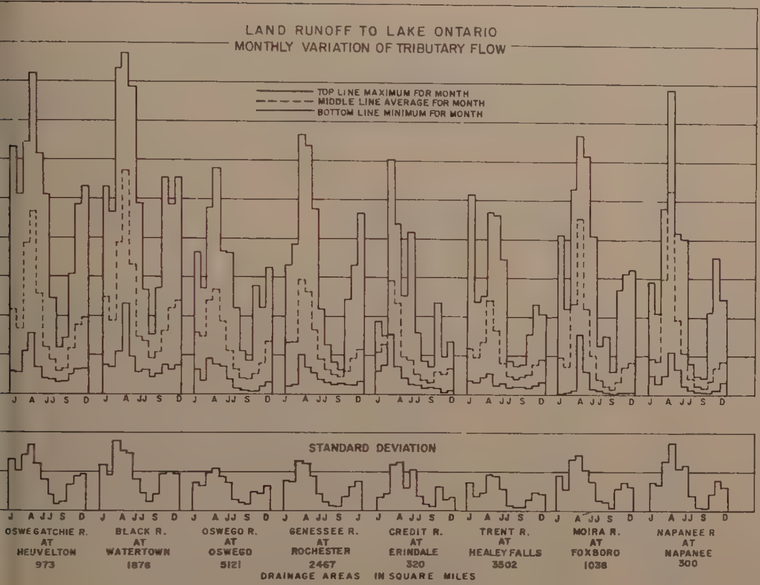


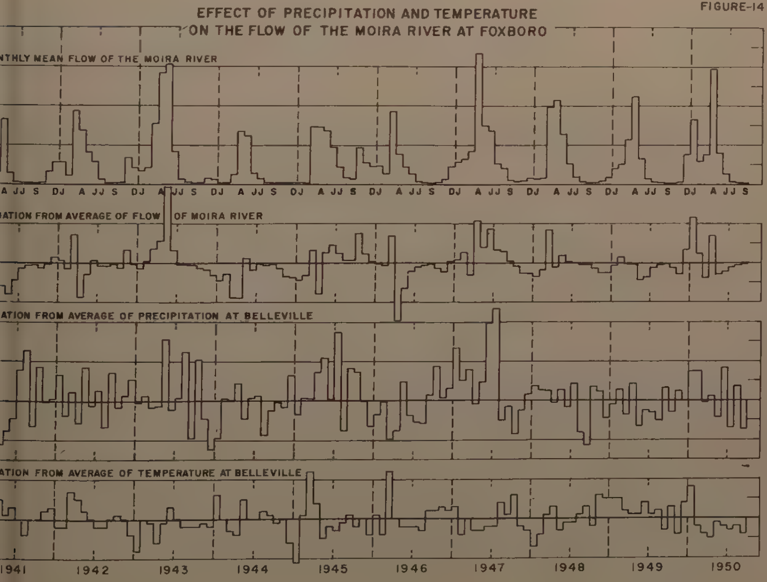
FIGURE-12



FIGURE–13



FIGURE–14



the river and it is believed that the deviations of the temperature and preci tation from average reflect conditions throughout the watershed with suffic accuracy for this study.

Although there are many interesting occurrences shown on this plate on three periods will be examined closely, These periods have been chosen t illustrate the factors influencing the variability of runoff which have been d cussed previously. The results of the examination can be summarized as follows:

Year	Month	Deviation from Average Precip: in:	Temp: °F	Runoff in:	Remarks
1947	May	+0.98		+0.80	Increases in the evaporation and
	June	+3.98		+1.75	transpiration losses reduce the
	July	+4.67		+0.66	effects of very much above avera precipitation.
1945	August	-1.49		+0.18	Decreased evaporation and trans-
	September	+1.61		+0.18	piration losses allow the increa
	October	+1.42		+1.52	rainfall of September and Octobe to be reflected in the runoff fo October.
1949-1950	November	+0.50	-4	-0.59	Accumulation of snow.
	December	-0.01	+4	+0.68	Some snow melt.
	January	+1.46	+8	+2.36	All snow melted.
	February	+1.47	-2	+0.48	Accumulation of snow.
	March	-0.06	-4	-0.79	Accumulation of snow.
	April	+0.20		+1.40	Melting of snow accumulated in February and March.

It may be concluded from the foregoing analysis of runoff variability tha the local drainage basin of Lake Ontario lies in a transition zone with respe to winter and spring runoff. To the north of the basin the snow cover is con tinuous throughout the winter so that the winter runoff is usually low and un form and the spring runoff high and irregular. South of the basin, continuou accumulation of snow is rare so that the runoff varies more directly with variations in precipitation. In the drainage basin of Lake Ontario itself, either of these conditions can occur depending on temperature variations during the winter months. It may be concluded that, because of the varia- bility of temperature during the winter, the use of snow surveys for the pre diction of runoff may have a limited use only.

Although instantaneous and daily peak flows are of little importance in a study of the hydrology of Lake Ontario, an indication of the flood potential o the drainage basin is of interest. Figure 16 illustrates a plot of the peak flows of the gauged tributaries against the drainage area. The envelope of peak flows and its equation are also shown on the plate. It is interesting to note that the peak quarter-monthly net local water supply lies slightly above the envelope of extreme flows for the tributaries. This may be explained b the fact that the period of record represented by net local water supply is 9 years which is much longer than the period for any of the tributaries of the basin. The maximum quarter-monthly net local water supply occurred in 1873.

FIGURE-15

AVERAGE DISTRIBUTION OF PRECIPITATION AND EVAPORATION ON LAKE ONTARIO 1934-1952

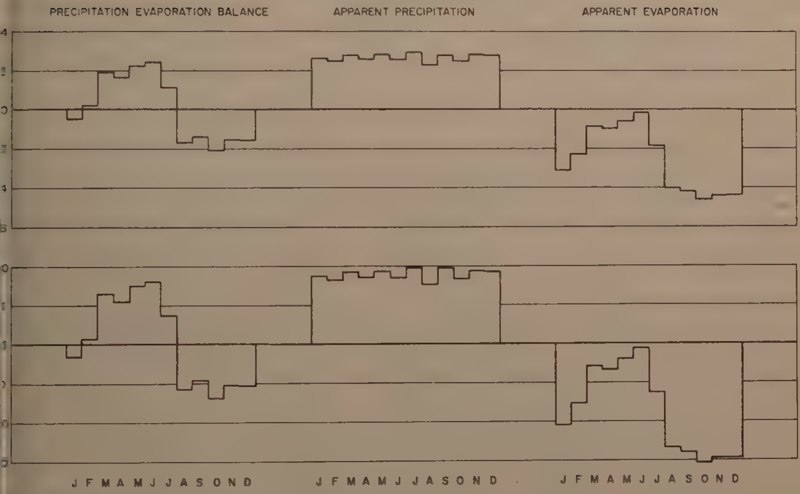
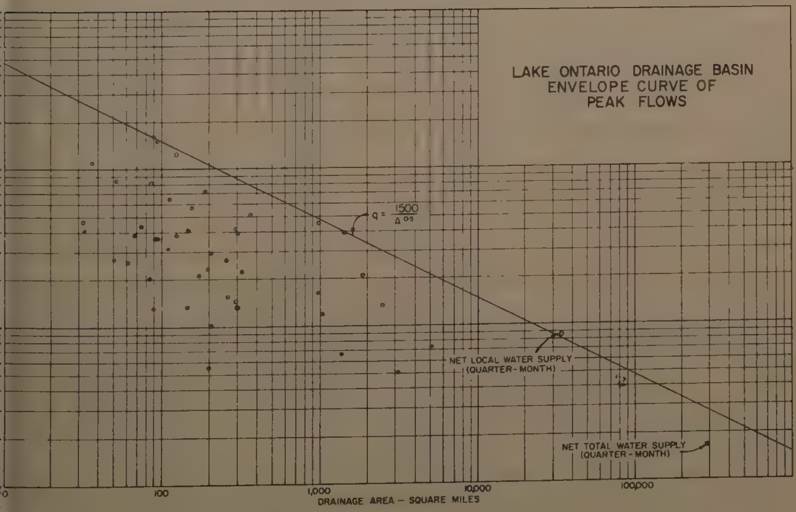


FIGURE-16



Precipitation-Evaporation Balance

The contribution of the surface area of Lake Ontario to the net local wat supply is derived from the precipitation falling on the surface as modified b the evaporation from the surface. Since there is no direct method of measu ing either precipitation or evaporation on such large lake surfaces the precipitation-evaporation balance can only be found from a water budget stu Such a study has been made for the period from 1934 to 1952 inclusive. Th total land runoff for each month has been derived by adding together all the measured inflows, with the exception of the Niagara River, and estimating t remainder of the land runoff on the basis of the known unit water yields. Th estimated land runoff was then subtracted from the net local water supply t obtain the precipitation-evaporation balance of the lake. Since it represent the difference between two large quantities, the precipitation-evaporation balance for any one month is subject to large percentage errors. However, the average for each month over 19 years shows the average distribution of the precipitation-evaporation balance with sufficient accuracy for the purposes of this study.

The average distribution of the precipitation-evaporation balance is show on Figure 15. This figure would indicate that the average contribution of th surface area of Lake Ontario to the net local water supply is negligible and that the only effect of the lake area is to increase the net local water supply during the seasons of high land runoff and decrease it during the seasons of low land runoff. The average cumulative effects of the seasonal increases and decreases are substantial, being of the order of 60,000 cubic feet per second for one month or three quarters of a foot on the lake.

There is no method of accurately analysing the two components of the precipitation-evaporation balance. However, by using the precipitation records of stations near the shore of the lake the apparent precipitation on the lake has been determined. By subtracting this from the precipitation-evaporation balance, the apparent evaporation was deduced.

It may be noted from Figure 15 that the average apparent precipitation varies very little throughout the year. The average apparent evaporation, however, varies considerably reaching its maximum value in October and it minimum value in June. It is relatively uniform from August through Decer ber.

Outflows and Water Levels of Lake Ontario

The hydrologic factors influencing the water supply to Lake Ontario have been given careful consideration. In the past the changes in water supply have resulted in changes in water levels and outflows from Lake Ontario bu because of the large storage capacity of the lake and the constricted outlet conditions the changes have been modified and delayed. Thus, although a range of one foot in the elevation of Lake Ontario can absorb 80,000 c.f.s. for one month a one foot change in the water level only changes the outflow by about 20,000 c.f.s.

The control section of Lake Ontario is in the St. Lawrence River downstream from Prescott, Ontario. Previous to the construction of the St. Lawrence Power Project, this was made up of a natural rock ledge across the river. During the past century, however, slight changes in this control section have resulted in overall changes in the levels of Lake Ontario. The water levels which will be used in further discussion are the recorded value

ted for these changes and for the diversions. During the winter, ice is ed at and above the control section restricting the outflow and causing ases in the water levels of the lake. Temporal changes in the outflows vater levels of Lake Ontario are caused by winds and seiches. Because location and greater depth, the effects of winds and seiches on Lake rio are not as great as on Lake Erie.

) obtain a more detailed knowledge of the interaction of the outflows from Erie and the net local water supplies and their effects on the water s of Lake Ontario in the past, a further investigation was made. It was assumed that the outflows from Lake Erie remained constant at their ige value but that the net local water supplies varied as they did in the These hypothetical net total water supplies were then routed through Ontario using standard routing procedures and taking into account the l effects of ice formation that had occurred. The same procedure was with another set of hypothetical supplies which were prepared by as- ng that the net local water supplies remained constant at their average but that the outflows from Lake Erie varied as they had in the past. The ts obtained were thus directly comparable with the recorded adjusted levels of the lake and are summarized in the following tabulation:

Monthly Mean Water Levels and Ranges of Stage

	Under Recorded Adjusted Conditions	Outflows from Lake Erie constant Net Local Water Supplies vary as in the past	Net Local Water Supplies constant Outflows from Lake Erie vary as in the past
ium Water of Lake io	248.93	248.67	247.86
ium Water of Lake io	242.84	244.11	244.03
um Range age	6.09	4.56	3.83
ge Annual of Stage	1.75	1.81	0.62

e marked decrease in the average annual range of stage, compared with ded adjusted conditions which would have occurred had the net local supplies been kept constant, indicates that the variations in the net water supply cause most of the annual variation in water levels. Con- ly, the slight increase in the average annual range of stage that would occurred had the outflows from Lake Erie been kept constant shows that asonal variation of the outflows from Lake Erie is not in phase with nal variations in the net local water supply. The effect of the long term ions in the outflow from Lake Erie is shown by the decrease in the num range of stage with respect to recorded adjusted conditions that have occurred had these been kept constant. The relatively greater de- e in maximum range with constant net local water supplies shows that

the variations of the net local water supplies have had a greater effect on th variation of water levels than have the variations in the outflow from Lake Erie. Since the variation of annual average water supplies is smaller for th net local water supplies than for the outflows from Lake Erie, it can be con cluded that the seasonal variations of the former are of considerable impor tance in producing the maximum range of stage.

It is difficult to evaluate the effect of the variations of the different sourc of water supply on the regulation of Lake Ontario. Critical water supply co ditions can vary in length from three months to three years. In the studies which have taken place it has been found that the critical recorded adjusted levels can be used as an indication of critical conditions for regulation. Wit this criterion and from the results of the foregoing routing study, it may be concluded that the variations in the net local water supply will cause most o the difficulties in regulating the outflows and water levels of the lake. It ca also be concluded that regulation of the upper Great Lakes which would mer ly smooth out the seasonal fluctuations in the outflows from Lake Erie woul not assist in the regulation of Lake Ontario although if they could be change radically to offset the seasonal variations of the net local water supply som improvement might result. A system of regulation which could smooth out the long term variations in the outflows from Lake Erie would benefit considerably the interests concerned with the regulation of Lake Ontario. Consideration of the vast volume of water involved in these long term variation indicates that an appreciable improvement would be very difficult to achiev

Forecasts of Net Total Water Supply

The regular seasonal variations in the outflows from Lake Erie and the delayed response of the long term variations in outflow to the variations in precipitation on the drainage basin of the upper Great Lakes indicate that it is possible to forecast the outflows from Lake Erie for a long period in advance with a considerable degree of accuracy. However, with the extreme variability of net local water supply and its dependence on variations of tem perature as well as of precipitation, accurate long term forecasts of this component could not be expected. With the net local water supply such an important factor in the regulation of Lake Ontario, it would seem impractic ble to devise a method of regulation that was based entirely on forecasts.

Forecasts may be of use, however, as a supplement to regulation. Such forecasts may be of short or long term. The accuracy of short-term forecasts of water supply would depend upon the accuracy of temperature and precipitation forecasts. Long term indications of future supplies may be de termined on a statistical basis utilizing the observed, but not invariable, te dency of the annual mean outflows of Lake Erie and the annual mean net loc water supplies to be high or low at the same time. Because forecasts of su ficient accuracy would be of too short a period and the long term indication would not be sufficiently accurate, the use of these techniques in the regulation of Lake Ontario would require limitations on the use of storage established by testing over the period of record.

SUMMARY

From the study of the hydrology of Lake Ontario which has been present it has been determined that the water supply to Lake Ontario has two princi

onents. The outflows from Lake Erie provide a very large and stable flow to Lake Ontario which increases or decreases predictably in re- e to long term changes in hydrologic conditions on the upper Great s. Although the ratio of the variations is small in terms of mean out- the long term variations do represent an extremely large volume of . The net local water supply, although small in proportion to the out- rom Lake Erie contributes more to the variation of both net total water y and water levels of Lake Ontario. Although the average seasonal dis- ion of precipitation for the drainage basin is very uniform other factors the net local water supplies to vary widely. These factors are the vari- in the areal distribution of precipitation on the local drainage basin, the nal variation of water losses, the effect of the winter temperature vari- on the accumulation of snow and the effect of the seasonal variations of ecipitation-evaporation balance on the lake which intensify the effect of nal variations of the land runoff. In addition to yielding an understanding water supplies to Lake Ontario the results of the study have indicated ne possibility of making accurate long range forecasts of water supply is remote. It has also been indicated in the study that the regulation of per Great Lakes in such a manner as to obtain an appreciable benefit to sts on Lake Ontario would be very difficult.

Journal of the

HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

RESISTANCE EXPERIMENTS IN A TRIANGULAR CHANNEL[a]

Ralph W. Powell,[1] M. ASCE and Chesley J. Posey,[2] M. ASCE

SYNOPSIS

. 400-ft adjustable-slope triangular flume was tested smooth and also :hened with small rectangular battens at various spacings. A comparison ıade of Manning's formula with some more recent suggestions, and the :city distribution, secondary currents, and other phenomena are reported riefly.

INTRODUCTION

'his paper reports the first phases of a comprehensive study of resistance ow in open channels being conducted at the Rocky Mountain Hydraulic oratory, Allenspark, Colorado since 1954. The major part of the support come from grants from the National Science Foundation, the first two of :h were received through the Colorado State University with Maurice L. rtson,[3] M. ASCE as co-director. Support has also come from the Water ources Division of the United States Geological Survey, the State Univer- of Iowa, the Ohio State University, the University of Kansas, and from 'unds of the Laboratory itself.

'he triangular shape of flume was adopted to maintain geometrical simi- y at all depths. Several others, such as Rogers,(5) Varwich,(9) Owen,(13) Moeller-Hartmann(15) have tested the same shape, but their work was indoors where the length was necessarily limited. The authors feel that a short flume it is very difficult to tell when the flow is at normal depth, e the upstream and downstream depths may be the same as nearly as can ıeasured, for quite a range of depths. (This point is discussed more fully

: Discussion open until October 1, 1959. To extend the closing date one month, a rritten request must be filed with the Executive Secretary, ASCE. Paper 2018 is art of the copyrighted Journal of the Hydraulics Division, Proceedings of the ımerican Society of Civil Engineers, Vol. 85, No. HY 5, May, 1959.

'resented at the October 1958 ASCE Convention in New York, N. Y.

ımeritus Prof. of Eng. Mechanics, Ohio State Univ., Columbus, Ohio.

'irector, Rocky Mountain Hydr. Lab. and Head, Dept. of Civ. Eng. State Iniv. of Iowa, Iowa City, Iowa.

'irector of the Research Foundation, Colorado State Univ., Fort Collins, olo.

later.) Therefore this flume was built outdoors and made 400 feet long. It is believed to be the longest channel systematically tested since Darcy and Bazin(1) and quite certainly the longest one of variable slope. As described later, the slope of this flume could be adjusted from nearly zero to slightly over 1.5%.

Design of the Tests

The purpose of an open-channel friction formula is to express the relationship between the channel shape and roughness, the discharge or velocity, the depth of flow, and the bottom slope for the condition known as steady uniform flow. Since the yearly cost of open channel construction in the United States undoubtedly totals many millions of dollars it behooves us to make sure that we evaluate this relationship as accurately as possible, introducing minor variables such as the temperature, if necessary, and even increasing the complexity of the computation if that would yield a better result.

Despite the wide use of open-channel friction formulas, it should be noted that the condition of steady uniform flow is seldom encountered, either in the field or in the laboratory. The depth of flow for steady uniform flow, known as the normal depth, is an abstraction, an extremely useful tool without which flow profiles in open channels could not be predicted with any satisfactory degree of reliability.

In determining normal depths in the laboratory, however, it is essential that steady uniform flow be attained just as closely as possible. Unsteady flow may differ imperceptibly from steady flow in all other respects but the depth of flow. In the present experiments, not only was the discharge introduced into the flume held as constant as possible, but the water-surface profiles were repeated as many times as necessary until they reached equilibrium, as far as that could be determined by the precision of setting and reading the point gages. For some conditions of flow, this would take twenty minutes or longer.

After the flow is steady, it is even more difficult to be sure that it is uniform. Backwater curves of the M1 and M2 type, corresponding to slightly different tailgate settings, may be very nearly parallel to the bottom, and yet have an appreciably different depth of flow. That the flow is uniform cannot safely be determined by reading gages at each end of the flume, for the error in setting and reading only two gages, especially if the flume is short, may seem to indicate uniform flow whereas actually the flow may be well along an M1 or M2 curve, and the apparently uniform depth far from the desired normal depth.

In lesser degree, the same thing is true for flow on steep slopes. Although the S2 and S3 backwater curves are not nearly as long as the M1 and M2 curves, there are apt to be standing waves on the surface that make accurate setting of point gages more difficult.

In determining the normal depths, therefore, profiles were obtained both below and above the normal depth, each profile for a steady flow condition. Fig. 1 shows a typical set of profiles for flow on a mild slope, and Fig. 2 a typical set of profiles for flow on a steep slope. The different curves in Fig. 1 are obtained from different settings of the tail gate at the lower end of the flume, and those of Fig. 2 from different settings at the entrance. Notice that in the 9:52 a.m. series of August 1, 1957 the normal depth of 2.475 feet is

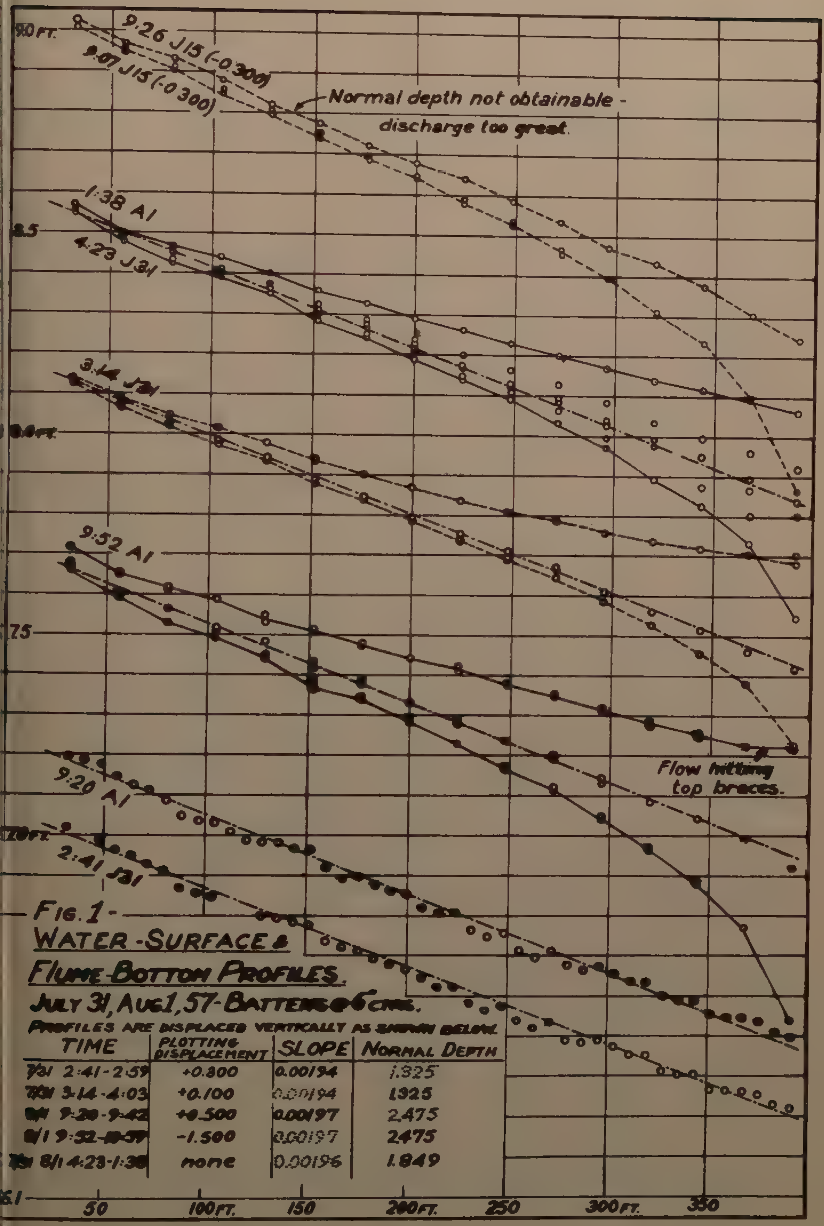


FIG. 1 - WATER-SURFACE & FLUME-BOTTOM PROFILES.
JULY 31, AUG 1, 57 - BATTENS @ 6 CMS.
PROFILES ARE DISPLACED VERTICALLY AS SHOWN BELOW.

TIME	PLOTTING DISPLACEMENT	SLOPE	NORMAL DEPTH
7/31 2:41-2:59	+0.800	0.00194	1.325
7/31 3:14-4:03	+0.100	0.00194	1.325
8/1 9:20-9:42	+0.500	0.00197	2.475
8/1 9:52-10:37	-1.500	0.00197	2.475
7/31 8/1 4:23-1:38	none	0.00196	1.849

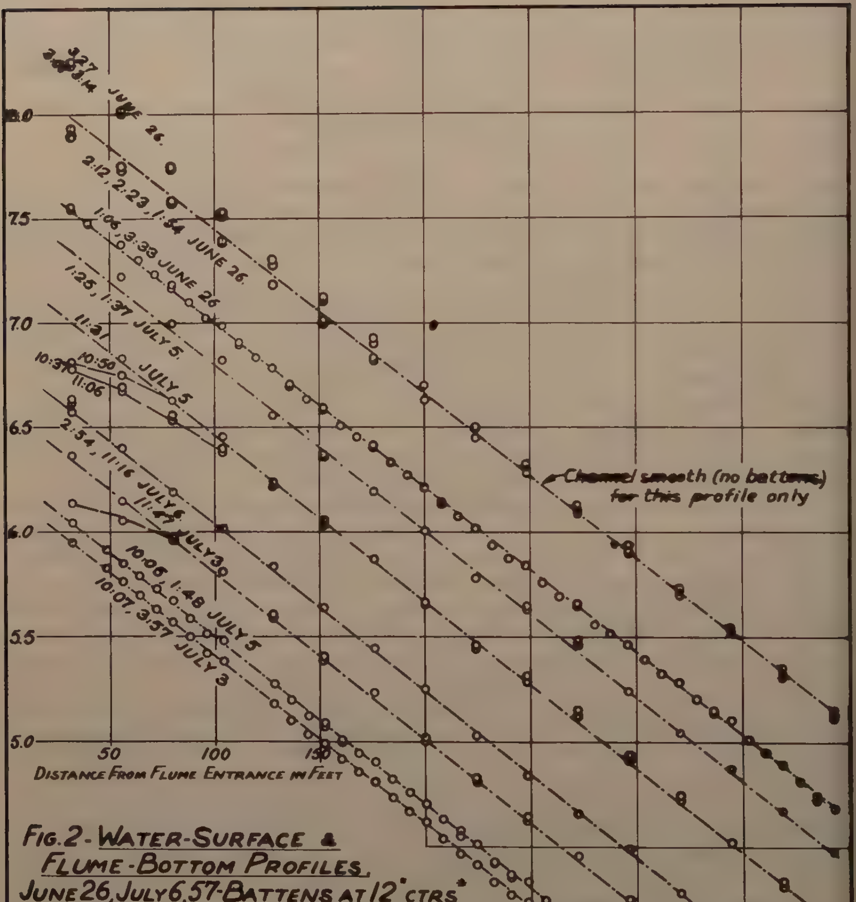


Fig. 2 - Water-Surface & Flume-Bottom Profiles, June 26, July 6, 57 - Battens at 12" ctrs.*

Profiles are displaced vertically as shown below

Time	Plotting Displacement	Slope	Normal Depth
6/26 1:06-1:30 & 3:38-3:58	+1.000	0.00786	1.45
6/26 1:54-2:34, 3:00-3:06	NONE	0.00786	1.45
7/3 10:07-10:28, 3:57-4:20	-0.600	0.00789	1.098
7/3 10:56-1:25	-1.300	0.00789	1.098
7/5 10:05-10:25, 1:48-2:02	-0.500	0.00797	1.863
7/5 10:37-11:45	-1.400	0.00797	1.863
7/5 1:25-1:44	-1.400	0.00797	2.200
7/6 11:16-11:22	-0.400	0.00797	0.437

* Except in one instance as noted on profile.

irly well determined by an M1 run, an M2 run, and two runs very near the ormal depth. This is about the maximum depth that could be obtained in the ume at the setting of that date, for no normal depth could be determined for e slightly greater discharge of 9:26 a.m., July 15, 1957.

The 1:54 - 3:06 run of June 26, 1957, shown in Fig. 2, gives a normal depth 1.45 feet which is about as great as could be determined at this roughness d slope. Of course, for the smaller depths on a steep slope, the S2 and S3 rves are comparatively short, the normal depth is soon reached, and it was sually sufficient to measure only the S2 curves starting from the critical epth.

When a test is being started in the field, with the depth for uniform flow t determined, the experimenter does not know whether the flow is tranquil shooting. To be sure, if it is well within either range the characteristics the flow will immediately show which one it is. Flow on the critical boun-ry range, however, does not have any well-defined characteristic except rface instability. Hence, whenever there was any question, profiles were ken with different headgate settings and also with different tailgate settings. is was not done with intent to explore the interesting backwater curves in is vicinity, but rather to establish the depth of uniform flow as reliably as ssible.

Description of the Apparatus

Water is diverted from the North St. Vrain Creek into a forebay, through metal flume, and into a weir box 42 feet long, 8 feet wide and 7.5 feet deep. e Fig. 3. The flow can be controlled at the point of diversion and there is emergency spillway for protection against high water in the forebay at the le of the metal flume, which latter can be barricaded against extreme high ter. Flow is stopped by inserting the needles at the point of diversion and sting leakage over the emergency spillway. It is started again by closing emergency spillway and removing enough needles to obtain approximately desired discharge. Adjustment to hold the flow steady is made by moving needles in a wasteway in the side of the weir box. Only a small amount discharge was permitted at this station, the operator of which could see a of temporary hook gages fastened inside the weir box, and adjusted to w deviations from the desired water level of only a few thousandths of a t.

Discharge from the weir spilled into a deep tumble bay from which it ered the variable-slope flume over a curved galvanized metal entrance tion. The radius of curvature was about 2 feet and the bottom of the ble bay was 6 feet below the bottom of the flume.

The variable-slope flume had a triangular cross-section with central angle t did not vary more than 0.0025 radian from ninety degrees. The bisector his angle was set vertical; in other winds, each side was set at 45° with horizontal. The walls of the flume were made of four-foot by eight-foot els of one-half inch fir plywood, exterior grade, stiffened at one-foot cen-s by fir two-by-fours. The two-by-fours along the bottom edge were off-slightly so that the reinforced panels nested and formed a tight joint, ch was held together by 1/4 inch bolts at one-foot centers. When the tops e fastened the correct distance apart by struts and heavy wire ties spaced wo foot eight inch centers, a rigid eight-foot section of flume was formed

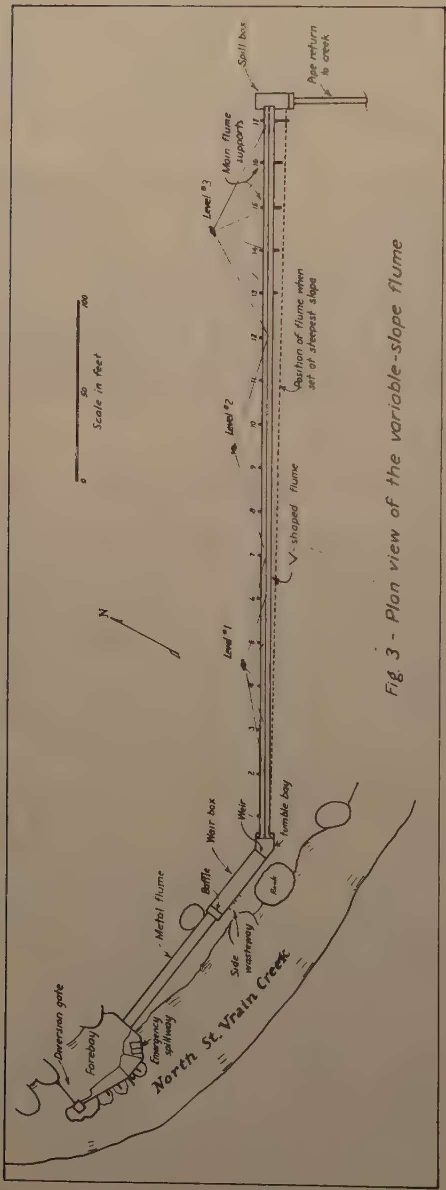


Fig. 3 - Plan view of the variable-slope flume

hich could be joined to its neighbors by splice plates. Fabrication was with hillips head wood screws.

The flume was supported on heavy timber bents at twenty-four foot centers. he upper surfaces of the supporting timbers were set to a common plane at n inclination of 45° with the vertical. Bearing blocks under the reaction stif-ners of the "bent-plate" girder formed by the flume sections rode up and own the 45° inclines, as controlled by jacks resting on the inclines. The ume was, therefore, approximately to a true grade whenever it was in align-ent. Final adjustment perpendicular to the incline was made by driving ingles under the bearing blocks. Before water was introduced additional earing blocks were inserted under the flume at the third-points between the clined supports, and wedged tightly above vertical logs carrying the load own to wooden plates set directly on the ground. To prevent tipping, the ends the stiffeners lying over the inclined supports were wired down. Since the pper flanges of the bent-plate girder did not have sufficient stiffness perpen-cular to the plane of the side, they had to be braced at the third points. This so prevented the flume from tipping.

Setting the flume accurately to a new slope required three days at first, but ith improved methods the time was cut down to little more than two days. he flume was quite stiff, and raising one jack only half an inch would lift it om supports 24 to 48 or even 72 feet on each side without causing distress noticeable distortions at the joints lifted. It is hoped that when the hydrau-c tests have been completed, the flume will still be in good enough condition permit structural tests of the bent-plate girder to be made.

The plywood panels were given a coat of penetrating sealer and one coat of rest green paint when fabricated at the hydraulic laboratory of Colorado ate University in Fort Collins early in the summer of 1955. When it was esired to start testing in 1956, it was found that the inner surface of the ume, which had been exposed to the weather, was beginning to check, lose int, and even suffer some ply separation. Since the bearing blocks under e intermediate stiffeners were not of oak, as had been specified but found be unavailable, and the intermediate splice plates not completely fastened e to a shortage of screws, the flume had developed a "sway-backed" con-tion with sags at the intermediate points ranging from 0.01 to 0.03 foot. ere were also slight local bulges (less than 0.02 foot) over the main sup-rts, caused by placing wedges under the stiffeners instead of under the aring blocks.

While the flume was being realigned and the missing screws inserted, it came apparent that the painted surface was suffering rapid deterioration. though the altitude is 8000 feet above sea level, day-time temperatures in e intense sunlight in the flume went as high as 130°F, while frost formed the surface at night. Several solutions to the problem were tested. That ich was finally adopted was to paint the entire interior surface with two ats of aluminum asphalt paint, using rollers and brushes. The resulting rface was fairly smooth, daytime temperatures were much lower, and the pid deterioration was stopped. Cracks in the surface and between panels ere sealed with asphalt-asbestos caulking compound, and, after painting, vered with black vinyl chloride adhesive tape. The tape alone would have en sufficient to seal the cracks, and perhaps better by itself as the asphalt d not have as much flexibility, and weakened the adhesion of the tape.

After a preliminary test with a maximum load of water, the bolts fastening e bottom together and the wires holding the top struts tight were tightened.

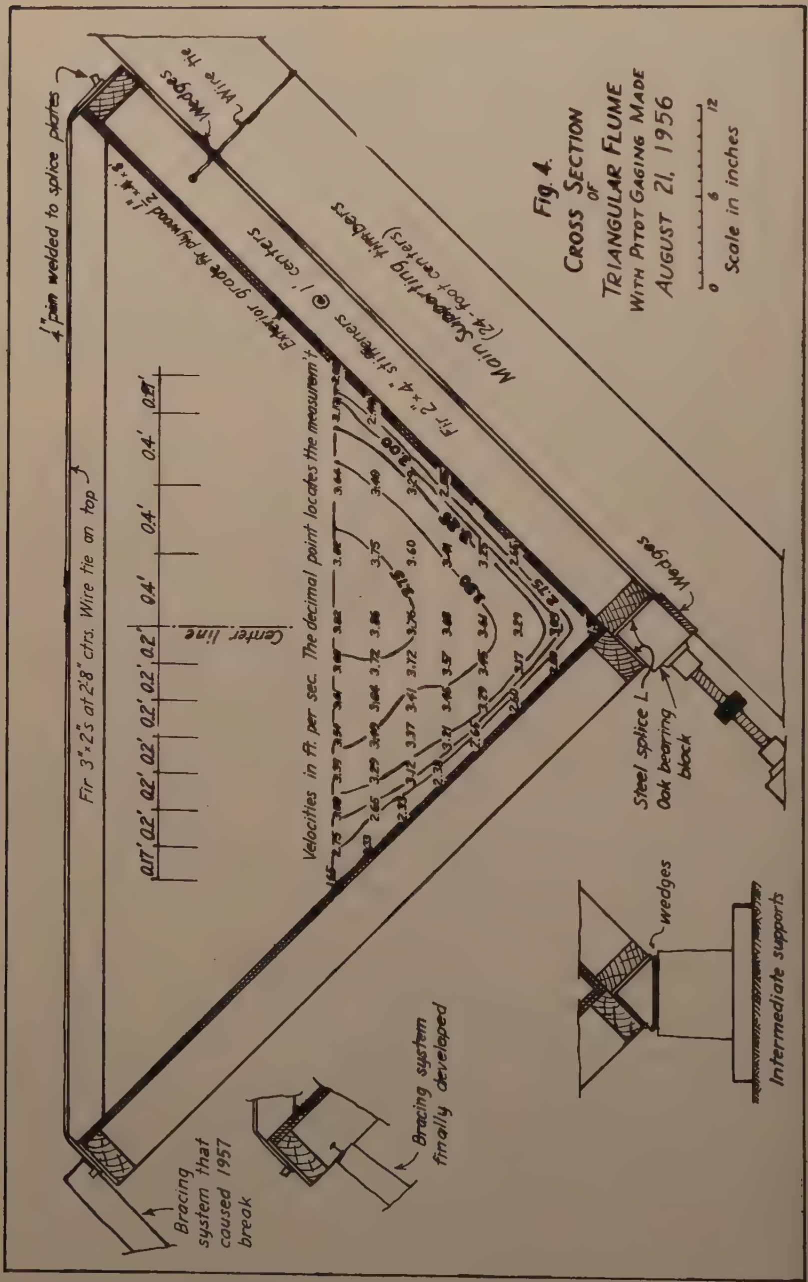


Fig. 4. CROSS SECTION OF TRIANGULAR FLUME WITH PITOT GAGING MADE AUGUST 21, 1956

ing subsequent tests no leaks greater than a few slow drips were observed. n July of 1957 a new method of bracing the flume that was being tried in ffort to cut down the time required for changing slope overstressed the , and during a run with the flume nearly full, 160 feet of it collapsed. The its of the design became apparent when it took only a little over two weeks ebuild, using mostly salvaged material. During the rebuilding greatly im- ved methods of fabrication were developed, and a simple and fast method racing and aligning the flume was discovered which solved that hitherto ersome problem.

pecially-built point gages were installed at twenty-four foot spacing along centerline of the flume. These gages were held vertical by adjustable orts fitted with grooved guides and a spring so that the gage could easily et with one hand.

n order to read the point gages which were set to indicate the water sur- and special level rods which were used to measure the bottom elevations, e levels were kept set up alongside the flume. Intermediate bench marks used to establish the instrument heights accurately, and in addition the gages midway between two instruments were usually read from both.

he "headgate" of the variable-slope flume, which had to be used when the city was supercritical, consisted of a vertical bulkhead with rounded bot- surface made from a log and sometimes from two or three additional logs ed so as to decrease further the entrance depth. It was fastened in at the r end of the flume, just at the end of the curved entrance section. Due to issymmetry of the tumble bay the surface waves below the headgate were unsymmetrical. Usually they could be reduced by fastening a baffle d at a location below the headgate that had to be determined by trial. The mmetrical surface waves seemed to disappear within a hundred feet of the nce in even the worst cases.

ne "tailgate" of the variable-slope flume was simply a set of needles n could be fixed in place as desired.

djustment of the headgate was required when the velocity was supercriti- nd adjustment of the tailgate when it was subcritical. In general, it was ractice to wait until the flow became steady, as indicated by the point s, before recording any readings, and to adjust both head and tail condi- if there was any doubt as to which should govern. Occasionally it was , upon repeating a set of profile readings, that changes had taken place, ating that the flow had still not become quite steady. The readings were ted until the slow changes disappeared.

Calculation of Flows

general flows of over 4 cfs were measured with a full-width sharp- ed weir and those smaller with a 90° V-notch weir.

e full-width weir was of stainless steel 24 in wide and 7.948 ft long. It bout 0.25 in thick but the upper edge was beveled at 30° to the vertical t for a flat top machined to 3/32 in width the same as Schoder and er's(3) Series D to P. At first every time the zero of the gage was de- ned, levels were taken at each foot along the crest. It was found that rth end was 0.004 ft higher than the south end, and the average of all adings was used to compute the discharges. But it was found that this ge was apt to be the same as the height at the middle, so that sometimes

only this was taken. The upstream face was as nearly vertical as it could b set with a carpenter's level, and the weir plate was held in place by counter sunk screws, so that the entire surface was smooth. The end of the weir bo below the stainless steel weir was of concrete and quite smooth and vertica but in the plane of the downstream face of the weir plate.

The 90° V-notch was cut in a sheet of 1/4 inch aluminum-painted temper masonite the same length as the full-width weir and 4 ft wide, which was placed with its lower part resting against the stainless steel weir plate and its upper part supported by a wooden frame. The bottom of the notch was about 0.3 in above the crest of the steel weir. The edge of the V was forme to the same shape as the full-width weir. It was found to keep its sharp edg very well. The ends and bottom were sealed with tape so as to make leakag negligible.

The head on the weir was read on a hook gage attached to a plank which was spiked to two trees beside the weir box. The hook gage well was a luci cylinder about 5 in. in diameter which was connected by about 4 ft of 3/4-in transparent hose to a short piece of half-inch pipe which projected from the side of the weir box at about 12 ft upstream from the weir. The inner end o this short pipe was filed to a plane and was flush with the inside of the weir box. The hook gage well was drained and refilled from the weir box whenev it was found that the water in the well was as much as 1.5°C warmer than th water flowing over the weir. The "zero" of the gage was determined by sett the hook to the cross hair of an engineer's level, and then reading a rod hel on the weir crest as mentioned above. For the V-notch a special target wa constructed which could be placed in the notch with contact along both edge rather than just at the vertex.

When a run was being made the hook gage was read every few minutes a recorded with the time. These readings were plotted against time, and fro this graph the average head for each run was determined. The median vari tion between maximum and minimum gage reading during a run was about 0.006 ft so that the error in the mean gage reading probably averaged about 0.001 ft. But there may have been another 0.001 ft error in the "zero readi

The flows from the V-notch were computed by the formula given by Lenz which reduces to

$$Q = K\,h^{2.5} \text{ with } K = 2.395 + \frac{2.994}{\underline{R}^{.165}\underline{W}^{.170}}$$

but not less than 2.485. $\underline{R}$ is $g^{0.5}h^{1.5}/\nu$ and $\underline{W}$ is $\rho g h^2/\sigma$ where σ is the surface tension in lb/ft. Since the laboratory is located at an elevation of 8000 ft above sea level, the value of g is less than at most laboratories. Upon the advice of Professor Weikko Aleksantri Heisknen of the Ohio State University, it was taken as 32.136 ft/sec^2.

It was originally planned to compute the flows over the full-width weir b Schoder and Turner's formula (D).(3) But when a velocity traverse was ma in the weir box opposite the hook gage piezometer to determine V_a and V_b was found that the V_b was quite a bit larger than V_a; that is, the highest ap proach velocity was near the bottom of the weir box. Schoder and Turner' data show that when V_b is larger than V_a, the flow is about the same as gi by Francis' simple formula, or sometimes even less, while formula (D) always makes it more. As described elsewhere,(17) a restudy of Schoder and Turner's data was made, leading to the formula

$$Q = (3.08 + 0.44\ h/P + 0.15\ r)(b - 0.003)(h + 0.003)^{1.5}$$

For our weir, b was 7.948 ft and P was 5.5 ft at the weir in 1956 and 1957, l this value was used for runs 1 to 108. However, the floor was not level l the average depth of the floor opposite the piezometer connection below weir crest was only 5.24 ft. Perhaps this should have been used for P for se runs but the change in Q would have been a very small fraction of one cent. Before work was started in 1958 a new floor was installed with 5.2 ft at the piezometer connection and 5.0 ft at the weir face. P was en as 5.2 for runs 109 on.

In the formula, r is the ratio of the velocity of approach above the level of crest to that below. Of course this was not measured for each run. From verses at five different heads and from Schoder and Turner's results, it ; decided that r could be taken as 0.435 h + 0.870. (This was for 1957 and 8 after additional baffling had been installed at the upper end of the weir . For the data of 1956 r was taken as 0.435 h + 0.199. This is different n the formula published in (16), so that the full-width discharges given in : publication have been revised.) All this leads to the following formulas discharge from the full-width weir:

1956 $\quad Q = 7.945\ (3.110 + 0.145h)\ (h + 0.003)^{1.5}$

1957 $\quad Q = 7.945\ (3.210 + 0.145h)\ (h + 0.003)^{1.5}$

1958 $\quad Q = 7.945\ (3.210 + 0.150h)\ (h + 0.003)^{1.5}$

Mr. R. E. Glover of the U.S.G.S. made some diffusion measurements by salt velocity method during the summer of 1956 and kindly supplied us 1 two values of the discharge which had also been measured by the full-th weir. The results are as follows:

Test No.	Q as measured by U. S. G. S.	R. M. H. L.	Difference R. M. H. L. - U. S. G. S.	
1	4.85 ± .02	4.88	0.03 cfs	= 0.6%
4	10.08 ± .11	9.96	-0.12 cfs	= -1.2%

Current meter gagings taken in the weir box to find r (as noted above) e the following rough checks of the weir formula:

Date	Value of Q in cfs by weir formula	by gaging	Difference	
ıg. 24, 1956	17.19	17.17	-0.02 cfs	0.1%
ıg. 7, 1957	10.22	11.49	+1.27 cfs	12.4%
ıg. 8, 1957	16.57	17.64	+1.07 cfs	6.5%
pt. 3, 1957	13.50	14.06	+0.56 cfs	4.2%
pt. , 1957	19.53	19.59	+0.06 cfs	0.3%
pt. , 1957	24.71	24.11	-0.60 cfs	2.4%

Pitot tube traverses taken in the flume gave the following rough checks:

Date	Value of α	Value of Q in cfs by weir formula	by traverse	Difference	
Aug. 21, 1956	1.05	6.68	6.65	+0.03 cfs	0.45
Aug. 10, 1957	1.09	14.51	14.88	+0.37 cfs	2.55
Aug. 15, 1957	1.07	14.44	14.38	-0.06 cfs	0.41
Aug. 23, 1957	1.03	17.40	17.79	+0.39 cfs	2.24
Aug. 27, 1957	1.06	17.90	17.76	-0.14 cfs	0.78
Aug. 30, 1957	1.04	23.72	23.39	-0.33 cfs	1.39

Determination of Bottom Slopes

For each slope and roughness a number of runs were made covering a wi range of discharges and a similarly wide range of depths of water in the flume. The weight of water in the flume varied from a few pounds per foot t as much as five hundred. Since the supporting system was necessarily subject to deformations from the varying load, and the heights of the main and intermediate supports differed along the length of the flume, it was deemed necessary to determine the bottom-profile of the flume under the conditions of the test runs, with the water flowing as near the normal depth as possible To permit this, special steel level rods were made which could be held in position even during the high velocity runs and read from the three levels se up along the length of the flume. Readings were taken at eight foot intervals along the bottom. The rod probably interfered slightly with the water-surfa profiles, but water surface-profiles were not taken while the rod was in plac and the rod was in the flume only part of the time. For some positions of th flume and levels, the supporting bents or other obstacles interfered with the line of sight, in which case the readings were simply omitted unless it was possible to include them by making an additional level setup without too muc delay.

Because of the time required, bottom-profile data were not taken for eve water-surface profile run. Usually bottom-surface profile data were taken corresponding to the highest and lowest runs of a given series. The difference between the mean bottom slopes and elevations determined at these ex tremes were usually negligible or very slight, so that the values for intermediate runs could be assumed to be the same as that of the nearer run, or interpolated between.

The profile data were plotted in pencil on matte-surfaced acetate sheets, using a scale of 20 feet to the inch horizontally and an adequate vertical sca depending upon the slope. After each of the observed elevations had been in in, using small circles centered on the plotted points, the straight line of be fit was determined by shifting a straight line drawn in ink on a separate she around under the acetate until its fit was judged to be as good as possible. Then a fine pencil line was drawn on the acetate above this inked line, usin a steel straightedge, and its elevation at each end of the flume determined, permitting computation of the slope.

In order to gain some idea of the accuracy of this graphical method of de termining bottom slopes Mr. Chintu Lai(19) made a study of the results

ained by 50 individuals from identical prints of the graph of data of 3:00 n. August 31, 1956 and compared it with the least squares value computed m the numerical data. He found the mean of the graphically-determined pes to be 0.008000, with standard deviation of 0.000030. Least squares ution gave a slope of 0.007998, with standard deviation (corresponding to scatter of the data) of 0.0000144. It seems reasonable to conclude that slopes were measured to within one-half of one per cent.

Determination of Normal Depths

Water-surface elevations obtained from level readings of the point gages unted at twenty-four foot intervals along the flume were plotted on the same tate sheets as the bottom-profiles, and after the plotted points were inked line best representing the normal depth was drawn by a similar technique, ing care to make it parallel to the appropriate bottom-profile line. For the 6 tests, this was done in the drawing room after the runs had been com- ted, but for the 1957 and 1958 tests the pencilled plots were made at the ne time the observations were taken, the slope and normal depth computa- s being made later. All of these drawings for the 1956 tests are repro- ed at reduced size in the R.M.H.L.'s Report No. 21, and it is expected that of the 1957 and 1958 profile drawings will be reproduced in a forthcoming ort. The results are summarized in Table I.

Brief History of Search for a Formula for Channel Resistance

The first formula for the resistance to flow in open channels that is still ognized is $V = C\sqrt{R\,S}$, proposed by Antione Chezy in 1775. He evidently oosed that C would be constant for any given channel, but this proved not e the case. In 1869 Ganguillet and Kutter presented a resume of practical- ll of the measurements of open channel resistance that had been made up hat time and from them derived the formula which bears their name. It very widely used for many years, but its cumbersomeness and the fact it is not dimensionally homogeneous has led to its replacement by other nulas.

n 1865 Bazin had proposed that the Chezy C could be expressed as $\overline{a + b/R}$ but in 1897 he suggested instead $C = \dfrac{A}{1 + \gamma/\sqrt{R}}$. Neither of e formulas seem to have been used in this country. In 1868 Philippe oard Gauckler, a French engineer, proposed the formula

$$V = \lambda\, R^{2/3}\, S^{1/2}$$

or some reason he limited it to slopes less than 0.0007. This formula ns to have attracted no attention at the time. Hagen proposed the same ula in 1881, without the limitation, and Manning(2) repeated it in 1889 out acknowledgement. But he was not satisfied with the formula and ad recommended a much more complicated one. In 1894 Crimp and es suggested $V = 124\, R^{2/3}\, S^{1/2}$ for sewers and water mains, a formula h is still used in England. In 1899 C. H. Tutton proposed to the Boston neering Societies the equation $V = \dfrac{1.54}{n} R^{2/3}\, S^{1/2}$ with n to be the same

as in Kutter's formula. Although this is obviously just as satisfactory as wha we today call the Manning formula, it seems to have never attracted attention It was not until books published by R. B. Buckley in 1911 and P. A. M. Parker in 1913 gave the formula

$$V = \frac{1.486\, R^{0.67}\, S^{0.50}}{n}$$

and for some strange reason called it Manning's formula, that it began to dis- place all other formulas for estimating channel resistance.

After the work of Prandtl, von Karman and Nikuradse had put a new light on the resistance to flow in pipes, and had shown the important difference be- tween smooth pipe and rough pipe flow, it was natural to ask if channels coulc not be treated similarly. Probably the first to propose formulas on this basi was Keulegan.(4) He was soon followed by Johnson,(7) Powell, (8, 10, 11) Kirschmer,(9) Robertson and Albertson(12) and others. However, none of these developed formulas that are satisfactory for universal use. In fact it became obvious that when a channel acts as smooth (that is any roughness which is present is entirely imbedded in the laminar boundary layer) the law of resistance must be different than it is when the boundary roughness governs.

There has been a tendency the last few years among the theoriticians to treat open channels as equivalent to a pipe with diameter four times the hy- draulic radius and then use the Weisbach formula and the Moody diagram for both pipes and channels. It is, of course, allowable to use the formula, but it must not be expected that f will be the same at the same Reynolds number, and the same wall roughness. The free surface and secondary currents woul be expected to increase the resistance.

A noteworthy treatment of the resistance to flow in rough channels is that of Morris.(14) Although the formulas he presents seem of little practical us and some of his generalizations too sweeping, he seems to have shown that there are at least three types of rough channel (and pipe) resistance, namely isolated-roughness, wake-interference, and quasi-smooth; and that it is foolish to suppose that an "equivalent sand grain size" with the Nikuradse rough pipe formula can be used to represent them all.

Discussion of Results

The results of the present tests are given in Table 1. It must not, of course, be assumed that values are correct to the last significant figure. Suppose that about 21 cfs were flowing in the flume at a depth of 2.40 ft and slope of 0.00200. This would be measured by the full-width weir with a head of about 0.850 ft. The measured head might be in error by 0.002 ft which makes a percentage error of 0.235%. Since Q varies as the 1.5 power of the head this indicates a percentage error of 0.35% in Q. But since the formula which is used in computing Q is probably not correct, it seems that the probable percentage error in Q might be as much as 1%. The slope may be assumed to have a probable error of 0.00001 or 0.5%. The error in deter- mining y_n might be as much as 0.005 ft or a probable percentage error of 0.21%. Since $f = 8\,S\,R\,g/V^2 = 90.9\,S\,y_n{}^5/Q^2$, the probable percentage error in f would be

$$\sqrt{0.5^2 + (5 \times 0.21)^2 + (2 \times 1.0)^2} = 2.3\%.$$

But now suppose that 0.043 cfs were flowing at a depth of 0.250 ft with a ɔe of 0.00045. With the same assumed error as before, the percentage or in the slope would be 2.22% and for the depth 2.00%. The flow would measured by the V-notch weir with a head of about 0.200 ft and probable centage error of 1.0%. Since the discharge varies as the 2.5 power of the d, this makes a probable percentage error in Q of 2.50%, or allowing for ertainty in the discharge formula, a total of say 3.0%. This makes the bable percentage error in

$$f = \sqrt{2.22^2 + (5 \times 2.0)^2 + (2 \times 3.0)^2} = 11.9\%.$$

s indicates that we might expect percentage errors in f in the range from 12%. When the flow is critical or shooting the error in measuring y_n ht be even more than assumed above. Since C and n vary inversely as square root of f, the expected percentage errors in them would be only ut half as much.

Results from Smooth Channel

When the values of f as found from the runs on the smooth flume (called n what follows) were compared with the values of f for a smooth pipe with same Reynolds number computed by Prandtl formula

$$\frac{1}{\sqrt{f}} = 2 \log (\mathbb{R}\sqrt{f}) - 0.80$$

ed f_p in what follows) the ratio $\Psi = f_o/f_p$ varied from 0.778 to 2.248. s 1-16 and 102-108 where the flow was definitely tranquil, with a maxi- ı Froude number (defined as $\frac{V}{\sqrt{g y_n/2}}$) of 0.85, gave an average Ψ of). In other words the f was only 93% as much as for a smooth pipe at same Reynolds number. This is very surprising. This reduces the for- ı

$$\frac{1}{\sqrt{f}} = 2.074 \log (\mathbb{R}\sqrt{f}) - 0.797$$

ranquil flow in smooth triangular channels. The average discrepancy of es of f computed by this formula from the observed values was 7.1%. e the square root of 1.071 is 1.035, this corresponds to an error of 3.5% or n. The average n was 0.0092 and the average discrepancy from this 0.00032 or 3.5%. Therefore, the logarithmic formula has no advantage the Manning formula for representing these data.

urning now to the smooth channel data in which the flow was critical or ting (Froude number more than 0.85), the best fit was obtained by making .15. This reduces to the equation

$$\frac{1}{\sqrt{f}} = 1.866 \log (\mathbb{R}\sqrt{f}) - 0.803$$

TABLE I - SUMMARY OF TEST RESULTS -

RUN NO.	DATE	TIME	BATTENS	WEIR	AVER. HEAD	TEMP.	SLOPE	y_n feet	Q c.f.s.	Froude No.	$R \cdot 10^{-3}$	Chezy C	f	Manning n
1	8-28-56	1:15 - 2:50	NONE	V	0.196	56°F	.000448	0.251	0.0427	0.34	18.4	107.4	0.0223	.0093
2	8-27-56	3:50 - 4:33	"	V	0.303	56°	.000468	0.370	0.1269	0.38	37.7	118.5	0.0183	.0089
3	8-27-56	1:54 - 3:05	"	V	0.502	55°	.000468	0.603	0.445	0.39	80.1	122.5	0.0171	.0094
4	8-27-56	11:19 - 1:14	"	V	0.826	54°	.000480	0.942	1.540	0.45	168.0	137.3	0.0136	.0090
5	8-24-56	4:08 - 4:40	"	⊔	0.331	56°	.000480	1.528	4.84	0.42	348	128.8	0.0155	.0104
6	8-24-56	10:37 - 1:01	"	⊔	0.767	53°	.000457	2.43	17.29	0.47	747	147.8	0.0118	.0098
7	8-17-56	3:35 - 5:10	"	V	0.184	57°	.000912	0.200	0.0364	0.51	20.5	113.3	0.0200	.0085
8	8-18-56	9:05 - 10:12	"	V	0.346	52°	.000875	0.368	0.1765	0.54	49.6	122.2	0.0172	.0087
9	8-20-56	2:54 - 3:15	"	V	0.542	54°	.000887	0.561	0.538	0.57	102.1	129.0	0.0155	.0088
10	8-20-56	1:25 - 2:12	"	V	0.773	53°	.000855	0.800	1.304	0.57	172.5	131.1	0.0150	.0092
11	8-20-56	9:20 - 10:45	"	V	1.226	49°	.000855	1.223	4.14	0.62	338	143.9	0.0124	.0090
12	8-21-56	8:50 - 10:42	"	⊔	0.412	48°	.000888	1.485	6.73	0.62	440	141.4	0.0129	.0094
13	8-21-56	2:56 - 5:05	"	⊔	0.893	55°	.001000	2.29	21.82	0.69	1031	146.3	0.0120	.0098
14	8-14-56	10:35 - 11:15	"	V	0.242	58°	.00198	0.232	0.0726	0.70	35.3	105.9	0.0229	.0092
15	8-14-56	12:35 - 1:33	"	V	0.446	57°	.00198	0.407	0.333	0.79	92.2	119.1	0.0181	.0090
16	8-14-56	2:13 - 2:55	"	V	0.704	59°	.00198	0.621	1.033	0.85	191.0	128.5	0.0159	.0090
17	8-11-56	11:13 - 12:01	"	⊔	0.271	54°	.00199	1.009	3.59	0.88	379	132.4	0.0147	.0095
	8-13-56	10:40 - 11:08												
18	8-13-56	11:40 - 2:13	"	⊔	0.473	59°	.00202	1.410	8.29	0.88	674	131.4	0.0149	.0101
19	8-13-56	2:55 - 3:46	"	⊔	0.942	61°	.00202	2.08	23.69	0.95	1344	142.1	0.0127	.0099
20	8-3-56	3:45 - 4:40	"	V	0.247	57°	.00376	0.202	0.0724	0.98	40.5	108.3	0.0219	.0088
21	8-6-56	4:05 - 4:13	"	V	0.292	55°	.00400	0.257	0.1152	0.86	48.5	91.5	0.0307	.0109
22	8-6-56	1:25 - 3:15	"	V	0.442	53°	.00400	0.383	0.325	0.89	88.7	95.2	0.0284	.0112
23	8-6-56	9:58 - 11:24	"	V	0.711	50°	.00400	0.563	1.057	1.11	188.5	118.2	0.0184	.0096
24	8-4-56	10:32 - 11:59	"	V	1.183	55°	.00396	0.875	3.78	1.32	460	141.1	0.0129	.0087
25	8-7-56	11:24 - 2:55	"	⊔	0.676	55°	.00400	1.447	14.26	1.41	1018	150.6	0.0113	.0088
	8-8-56	9:39 - 9:52				49°								
26	8-8-56	11:10 - 11:25	"	⊔	1.120	52°	.00401	1.994	30.94	1.37	1604	146.4	0.0120	.0096
27	8-31-56	1:29 - 2:17	"	V	0.374	52°	.00799	0.262	0.214	1.52	85.5	114.7	0.0196	.0087
28	8-31-56	3:35 - 4:36	"	V	0.536	52°	.00799	0.370	0.524	1.57	149.9	118.4	0.0184	.0090
29	9-1-56	9:30 - 10:35	"	V	0.706	47°	.00794	0.481	1.041	1.62	205	122.4	0.0172	.0090
30	9-1-56	11:04 - 12:32	"	V	1.222	50°	.00794	0.767	4.10	1.99	545	150.2	0.0114	.0080
31	9-3-56	11:36 - 2:47	"	⊔	0.516	53°	.00794	1.096	9.46	1.88	906	142.0	0.0128	.0090
32	9-4-56	10:45 - 3:10	"	⊔	0.678	55°	.00801	1.285	14.32	1.91	1206	143.8	0.0124	.0091
[illegible]	[illegible]	[illegible]	[illegible]	⊔	0.826	[illegible]	.00786	1.45	19.97	1.97	1905	149.6	0.0115	.0089

40	7-5-57	1:25-1:44	"	⊔	1.154	9.7°	.00800	2.200	33.39	1.16	1494	87.4	0.0337	.0163
41	7-9-57	9:50-10:25	6"-CTRS	V	0.402	7.3°	.00797	0.385	0.258	0.70	62.3	52.9	0.0919	.0202
42	7-9-57	11:02-11:18	"	V	0.547	8.2°	.00797	0.514	0.553	0.73	102.3	54.9	0.0853	.0203
43	7-9-57	11:39-11:53	"	V	0.858	8.5°	.00798	0.748	1.695	0.87	217.6	66.0	0.0590	.0180
44	7-9-57	1:18-2:05	"	V	1.229	9.8°	.00801	1.064	4.16	0.89	390.6	67.0	0.0573	.0189
45	7-9-57	3:37-4:32	"	⊔	0.479	10.5°	.00800	1.430	8.72	0.89	619.8	67.0	0.0573	.0198
46	7-10-57	9:15-10:50	"	⊔	0.701	7.0°	.00800	1.766	15.54	0.94	811.3	70.5	0.0517	.0195
47	7-10-57	11:22-12:00	"	⊔	0.840	7.6°	.00800	1.972	20.49	0.94	971.8	70.5	0.0517	.0198
48	7-10-57	1:23-2:38	"	⊔	1.167	8.3°	.00797	2.381	33.97	0.97	1363	73.1	0.0481	.0198
49	8-2-57	2:22-3:00	"	V	0.242	13.7°	.00194	0.303	0.0737	0.36	26.98	55.7	0.0828	.0183
50	8-2-57	1:47-1:55	"	V	0.358	13.1°	.00194	0.437	0.1937	0.46	48.35	58.6	0.0749	.0186
51	8-2-57	11:45-12:16	"	V	0.533	11.8°	.00194	0.626	0.518	0.42	87.28	63.8	0.0632	.0181
52	8-2-57	10:46-11:20	"	V	0.786	11.6°	.00194	0.896	1.361	0.45	159.2	68.4	0.0549	.0179
53	7-31-57	3:14-4:03	"	⊔	0.282	12.8°	.00194	1.325	3.93	0.49	321.3	74.3	0.0466	.0176
54	7-31-57	4:23-5:01	"	⊔	0.509	13.3°	.00196	1.849	9.56	0.51	567.2	78.1	0.0421	.0177
	8-1-57	1:47-2:29	"											
55	8-1-57	9:52-10:59	"	⊔	0.854	10.9°	.00197	2.475	21.01	0.54	873.4	82.6	0.0377	.0176
57	8-6-57	2:37-2:48	3"-CTRS	V	0.230	13.9°	.00193	0.311	0.0651	0.30	23.35	46.2	0.1207	.0222
58	8-6-57	1:32-2:11	"	V	0.349	13.8°	.00194	0.448	0.1819	0.34	45.12	51.7	0.0961	.0211
59	8-6-57	11:30-12:01	"	V	0.496	12.9°	.00195	0.617	0.434	0.36	76.4	55.3	0.0841	.0209
60	8-6-57	10:51-11:10	"	V	0.740	12.1°	.00196	0.889	1.171	0.39	140.0	59.6	0.0723	.0205
61	8-5-57	4:56-5:07	"	V	1.056	12.4°	.00197	1.240	2.85	0.42	242.8	63.0	0.0650	.0205
	8-6-57	10:03-10:27	"			11.5°								
62	8-7-57	11:56-12:15	"	⊔	0.236	11.9°	.00196	1.295	3.01	0.39	245.8	59.9	0.0718	.0218
63	8-7-57	11:05-11:34	"	⊔	0.459	11.8°	.00196	1.870	8.18	0.43	461.3	65.0	0.0610	.0214
64	8-7-57	9:57-10:47	"	⊔	0.730	10.9°	.00196	2.411	16.53	0.46	705.3	69.6	0.0529	.0208
65	8-13-57	11:50-12:05	"	V	0.226	12.7°	.00098	0.341	0.0624	0.23	19.76	49.4	0.1054	.0212
66	8-13-57	10:43-11:15	"	V	0.372	11.5°	.00098	0.544	0.213	0.24	40.94	52.4	0.0936	.0215
67	8-13-57	9:18-10:05	"	V	0.519	10.2°	.00100	0.734	0.486	0.26	66.78	56.0	0.0820	.0212
68	8-12-57	4:39-4:59	"	V	0.660	12.1°	.00100	0.907	0.879	0.28	103.1	59.7	0.0721	.0206
69	8-12-57	3:18-3:42	"	V	0.858	12.0°	.00102	1.164	1.695	0.29	154.3	62.9	0.0650	.0210
70	8-12-57	11:58-12:45	"	V	1.162	13.1°	.00102	1.547	3.62	0.30	255.4	64.1	0.0626	.0210
71	8-12-57	2:29-2:52	"	V	1.311	12.0°	.00102	1.703	4.89	0.32	304.3	68.0	0.0556	.0201
72	8-13-57	4:08-4:45	"	⊔	0.330	14.0°	.00102	1.734	4.98	0.31	321.0	66.2	0.0587	.0207
73	8-12-57	9:02-10:02	"	⊔	0.468	10.9°	.00103	2.120	8.41	0.32	408.1	67.4	0.0566	.0210
74	8-9-57	4:52-5:09	"	⊔	0.678	14.2°	.00103	2.585	14.77	0.34	641.8	72.0	0.0496	.0203
	8-13-57	3:10-3:44												
75	8-20-57	3:45-4:17	6"-CTRS	V	0.249	10.0°	.00101	0.363	0.0788	0.25	21.78	52.5	0.0933	.0201
76	8-20-57	3:00-3:22	"	V	0.363	10.0°	.00101	0.500	0.201	0.28	40.32	60.2	0.0709	.0185

CONCL'D ON NEXT PAGE.

TABLE I (CONCL'D)

RUN NO.	DATE	TIME	BATTENS	WEIR	AVER. HEAD	TEMP.	SLOPE	y_n feet	Q c.f.s.	Froude No.	$R.10^{-3}$	Chezy C	f	Manning n
77	8-20-57	1:42-2:23	6"-CTRS	V	0.511	9.9°C	.00101	0.677	0.467	0.31	65.99	65.8	0.0594	.0179
78	8-20-57	11:30-12:17	"	V	0.713	10.0°	.00101	0.912	1.068	0.34	117.4	71.1	0.0509	.0173
79	8-20-57	10:42-11:03	"	V	1.044	10.0°	.00101	1.308	2.77	0.35	212.4	74.9	0.0458	.0174
80	8-15-57	12:06-12:22	"	⊔	0.282	12.2°	.00102	1.514	3.93	0.35	276.8	73.4	0.0477	.0182
81	8-20-57	9:30-10:23	"	V	1.301	10.0°	.00101	1.598	4.80	0.37	301.2	78.7	0.0415	.0172
82	8-15-57	11:15-11:46	"	⊔	0.494	12.1°	.00102	2.064	9.14	0.37	470.9	78.6	0.0416	.0179
83	8-15-57	10:01-10:48	"	⊔	0.666	11.2°	.00102	2.449	14.38	0.38	608.7	80.7	0.0395	.0180
84	8-22-57	2:11-2:33	12"-CTRS	V	0.272	13.8°	.00102	0.355	0.0984	0.33	30.8	69.0	0.0540	.0152
85	8-22-57	11:35-12:13	"	V	0.358	12.4°	.00102	0.453	0.1937	0.35	45.8	73.9	0.0471	.0148
86	8-22-57	10:40-11:14	"	V	0.507	11.4°	.00102	0.623	0.458	0.37	76.7	79.0	0.0412	.0147
87	8-22-57	9:03-10:04	"	V	0.701	9.7°	.00102	0.842	1.024	0.39	120.9	82.9	0.0374	.0146
88	8-21-57	3:52-4:38	"	V	0.968	11.6°	.00102	1.138	2.29	0.41	210.9	87.2	0.0338	.0146
89	8-21-57	2:16-3:13	"	V	1.275	11.2°	.00102	1.477	4.56	0.43	320.0	90.6	0.0313	.0147
90	8-23-57	10:17-10:50	"	⊔	0.337	12.0°	.00103	1.567	5.13	0.42	347.2	87.4	0.0337	.0154
91	8-23-57	9:09-9:59	"	⊔	0.504	11.1°	.00103	1.962	9.41	0.44	496.2	91.5	0.0307	.0153
92	8-22-57	4:37-5:28	"	⊔	0.755	13.9°	.00104	2.465	17.40	0.46	787.4	95.1	0.0284	.0153
93	8-28-57	2:40-3:12	24"-CTRS	V	0.257	12.7°	.00102	0.316	0.0855	0.38	29.2	80.2	0.0400	.0129
94	8-28-57	12:08-12:30	"	V	0.408	11.0°	.00102	0.482	0.267	0.41	57.1	87.0	0.0340	.0127
95	8-28-57	10:52-11:25	"	V	0.535	10.1°	.00102	0.617	0.523	0.44	85.3	92.1	0.0303	.0125
96	8-28-57	9:20-9:51	"	V	0.717	9.5°	.00103	0.812	1.082	0.45	131.7	95.7	0.0281	.0126
97	8-27-57	4:22-4:44	"	V	0.981	12.3°	.00103	1.089	2.37	0.48	232.5	100.3	0.0256	.0126
98	8-26-57	3:30-4:18	"	⊔	0.307	12.9°	.00103	1.402	4.46	0.48	345.4	100.4	0.0255	.0132
99	8-27-57	3:00-4:01	"	V	1.282	12.2°	.00103	1.411	4.62	0.49	348.9	102.3	0.0246	.0129
100	8-26-57	1:15-2:41	"	⊔	0.534	12.9°	.00103	1.898	10.28	0.52	588.4	108.6	0.0218	.0128
101	8-26-57	10:01-11:32	"	⊔	0.766	10.9°	.00103	2.355	17.80	0.52	777.5	111.9	0.0205	.0132
102	8-29-57	3:55-4:03	NONE	V	0.223	10.3°	.00103	0.250	0.0604	0.48	24.4	101.2	0.0251	.0098
103	8-29-57	3:09-3:42	"	V	0.375	10.3°	.00103	0.394	0.217	0.54	55.7	116.7	0.0189	.0092
104	8-29-57	2:26-2:45	"	V	0.526	10.4°	.00103	0.532	0.503	0.61	95.9	127.7	0.0158	.0088
105	8-29-57	1:15-2:01	"	V	0.698	10.3°	.00103	0.705	1.011	0.60	144.8	126.7	0.0160	.0093
106	8-29-57	10:41-11:24	"	V	0.949	9.6°	.00103	0.937	2.18	0.64	230.5	134.3	0.0143	.0092
107	8-29-57	9:29-10:17	"	V	1.302	9.3°	.00103	1.269	4.81	0.66	373.1	139.1	0.0133	.0094
108	8-30-57	11:41-11:58	"	⊔	0.926	10.1°	.00103	2.332	23.79	0.71	1027	150.3	0.0114	.0096
109	7-3-58	11:44-12:27	"	⊔	0.255	12.8°	.01150	0.670	3.38	2.29	547	144.3	0.0124	.0081
110	7-3-58	10:33-11:25	"	⊔	0.384	11.8°	.01150	0.875	6.25	2.18	754	136.8	0.0137	.0089
111	7-1-58	2:08-4:14	"	⊔	0.544	14.0°	.01148	1.057	10.58	2.30	1120	144.6	0.0123	.0087
112	7-2-58	11:12-2:16	"	⊔	0.856	11.5°	.01149	1.375	21.11	2.37	1524	149.3	0.0115	.0088
[illegible]	[illegible]	[illegible]	[illegible]	⊔	1.036	11.0°	.01150	1.577	28.32	2.26	1852	142.3	0.0127	.0095

120	7-7-58	10:38-11:12	"	⊔	0.854	10.9°	.01154	1.632	21.04	1.54	1325	96.8	0.0274	.0140
121	7-7-58	9:46-10:22	"	⊔	1.048	9.9°	.01154	1.850	28.82	1.54	1556	96.9	0.0274	.0143
122	7-9-58	2:25-2:48	12"-CTRS	V	0.466	14.5°	.01148	0.404	0.371	0.89	104	56.0	0.0816	.0191
123	7-9-58	11:35-11:50	"	V	0.776	13.5°	.01148	0.641	1.318	1.00	227	62.9	0.0650	.0184
124	7-9-58	10:40-11:02	"	V	1.222	12.9°	.01148	0.944	4.10	1.18	472	74.3	0.0465	.0166
125	7-10-58	3:50-4:18	"	⊔	0.286	15.0°	.01148	0.960	4.02	1.11	481	69.8	0.0526	.0178
126	7-10-58	3:15-3:33	"	⊔	0.490	15.4°	.01150	1.260	9.03	1.26	831	79.5	0.0407	.0163
127	7-10-58	2:10-2:35	"	⊔	0.691	15.5°	.01151	1.540	15.22	1.29	1150	81.1	0.0392	.0166
128	7-9-58	4:41-4:55	"	⊔	0.992	14.4°	.01151	1.875	26.48	1.37	1595	86.2	0.0346	.0161
129	7-10-58	10:45-11:20	"	⊔	1.164	13.1°	.01152	2.206	33.90	1.17	1676	73.4	0.0476	.0194
130	7-18-58	11:13-11:35	"	V	0.542	12.9°	.01520	0.445	0.540	1.02	132	55.8	0.0827	.0196
131	7-18-58	9:55-10:25	"	V	0.835	12.1°	.01520	0.633	1.583	1.24	266	67.7	0.0560	.0171
132	7-18-58	8:56-9:28	"	V	1.108	12.0°	.01520	0.827	3.211	1.29	413	70.5	0.0518	.0177
133	7-17-58	2:17-2:36	"	⊔	0.355	14.5°	.01520	1.020	5.55	1.32	617	72.1	0.0495	.0174
134	7-17-58	1:30-1:54	"	⊔	0.494	14.3°	.01520	1.210	9.14	1.42	850	77.4	0.0429	.0167
135	7-17-58	11:50-12:06	"	⊔	0.635	14.0°	.01520	1.380	13.38	1.48	1086	81.6	0.0393	.0162
136	7-17-58	11:10-11:31	"	⊔	0.784	14.0°	.01520	1.551	18.46	1.54	1331	84.0	0.0364	.0160
137	7-16-58	3:49-4:23	"	⊔	1.049	14.3°	.01528	1.786	28.86	1.68	1820	92.1	0.0307	.0149
138	7-17-58	9:50-10:20	"	⊔	1.217	12.4°	.01528	2.004	36.32	1.59	1941	86.9	0.0340	.0161
139	7-19-58	8:55-9:31	24"-CTRS	V	0.588	11.5°	.01523	0.425	0.661	1.40	163	76.5	0.0439	.0142
140	7-18-58	4:41-5:08	"	V	0.803	13.4°	.01523	0.582	1.436	1.39	272	75.7	0.0448	.0151
141	7-18-58	3:48-4:17	"	V	1.116	13.4°	.01523	0.765	3.27	1.59	471	87.1	0.0339	.0137
142	7-21-58	2:50-3:10	"	⊔	0.399	12.0°	.01523	1.015	6.62	1.60	697	87.5	0.0336	.0144
143	7-21-58	2:12-2:28	"	⊔	0.539	12.1°	.01523	1.198	10.43	1.66	928	90.5	0.0314	.0142
144	7-21-58	1:22-1:41	"	⊔	0.748	12.1°	.01523	1.440	17.18	1.72	1270	93.4	0.0290	.0141
145	7-21-58	11:39-11:58	"	⊔	0.941	12.0°	.01523	1.657	24.42	1.72	1564	94.1	0.0290	.0134
146	7-21-58	9:49-10:26	"	⊔	1.204	10.1°	.01523	1.906	35.72	1.78	1882	97.0	0.0273	.0143
147	7-24-58	11:45-12:17	NONE	V	0.354	14.1°	.01524	0.255	0.1883	1.43	82.9	78.2	0.0421	.0127
148	7-24-58	11:00-11:27	"	V	0.699	13.6°	.01524	0.440	1.015	1.97	255	107.6	0.0222	.0101
149	7-24-58	10:21-10:34	"	V	1.028	12.8°	.01524	0.610	2.66	2.28	473	124.7	0.0165	.0092
150	7-24-58	9:33-9:48	"	V	1.287	12.3°	.01524	0.756	4.67	2.34	659	127.8	0.0157	.0093
151	7-23-58	4:21-4:37	"	⊔	0.400	15.8°	.01524	0.856	6.85	2.51	934	137.2	0.0136	.0088
151	7-23-58	3:49-4:08	"	⊔	0.509	15.8°	.01524	0.985	9.57	2.53	1163	138.3	0.0134	.0092
153	7-23-58	3:15-3:33	"	⊔	0.680	15.9°	.01522	1.130	14.85	2.77	1566	151.3	0.0112	.0085
154	7-23-58	2:32-2:52	"	⊔	0.839	15.5°	.01522	1.314	20.44	2.53	1776	138.2	0.0135	.0093
155	7-23-58	11:40-11:59	"	⊔	1.007	13.8°	.01522	1.470	27.10	2.58	2050	141.0	0.0129	.0094
156	7-22-58	4:05-4:30	"	⊔	1.117	14.1°	.01522	1.624	34.17	2.52	2337	137.6	0.0136	.0098
157	7-23-58	10:33-11:00	"	⊔	1.286	12.6°	.01522	1.698	38.79	2.59	2482	141.9	0.0128	.0097

for shooting flow in smooth triangular channels. The average percentage discrepancy of values of f computed by this formula from the observed values was 11.2% which corresponds to an error of 5.5% in V or n. The average n was 0.0094 and the average discrepancy from this was 6.8%, so that the Manning formula does not fit these data as well as the logarithmic. But the difference is not great, and neither formula gives satisfactory agreement. The explanation seems to be that when the flow is critical or shooting the surface is so irregular that there is too much error in measuring the depth.

Results from Rough Channel

The roughness used in the experiments described in this paper consisted of rectangular strips of redwood 3/16 in. thick by 3/8 in. wide tacked to the sides of the flume perpendicular to the flow. They stopped about 1/2 in. short of the bottom, so that the flume could drain dry after the day's runs were finished, but it was felt that this would have very little effect on the resistanc The strips had been sanded on the three exposed surfaces and the 3/16 in. dimension did not vary more than a few thousandths of an inch. The strips were used without any surface treatment except that they were soaked in wate before being applied to avoid their buckling out between brads when the water was turned on. Four longitudinal spacings were tested, 24 in. center to cente 12 in., 6 in., and 3 in. This last required 3200 battens. The longitudinal spacing center to center in feet is represented by the symbol λ as in Morris' paper.(14)

In searching for a formula to represent the rough channel results it was found necessary to divide the tests into two groups - those in which the flow was definitely tranquil (the Froude number less than 0.70), and those in which the flow was critical or shooting (Froude number 0.70), and those in which the flow was critical or shooting (Froude number 0.70 or more). It should be noted that these Froude numbers do not include the velocity distribution factor α.

For the tranquil case three formulas were tested. The first was Manning' It gave the following values of n:

Batten spacing	Value of n	Average discrepancy
24 in.	0.0128	1.6%
12 in.	0.0150	2.1%
6 in.	0.0180	2.5%
3 in.	0.0210	1.8%

The weighted average discrepancy in values of n for these 52 runs was 2.05%

The same runs were computed on the basis of Nikuradse's formula for rough pipe:

$$\frac{1}{2\sqrt{f}} = \log(r_o/\epsilon) + 0.87$$

Substituting 2 R for r_o this reduces to

$$\frac{1}{\sqrt{f}} = 2 \log R - 2 \log \epsilon + 2.342$$

rom this formula ϵ was computed for each run and the average value for ach spacing and the average percentage discrepancy in f computed by using is ϵ was found as follows:

Batten spacing	Average ϵ	Average % discrepancy
24 in.	0.00459 ft	2.57
12 in.	0.0126 ft	3.42
6 in.	0.0343 ft	3.67
3 in.	0.0694 ft	6.46

e weighted average percentage error in f was 4.40% which corresponds to 2.18% average error in n or V.

A third formula tried is based on Morris' proposal for isolated-roughness w which for our case reduces to

$$\psi = 1 + 67.2\, C_D\, h/\lambda\ ,$$

with h = 3/16 in. = 1/64 ft,

$$\psi = 1 + 1.05\, C_D/\lambda$$

lues of f_p and ψ were tabulated for each run and values of ψ plotted against λ as in Fig. 5. It was found that the equation

$$\psi = 4.43 - 0.145\,(4.88 - 1/\lambda\,)^2$$

presented the data fairly well. (This indicates that the maximum value of would occur when $\lambda = 0.2049$ ft = 2.46 in. or 13.1 times the batten height. is agrees quite well with Johnson's results. See Fig. 14 on page 558 of erence (8). The formula is not expected to hold beyond the maximum.) lues of f were compute for all 52 runs by this formula and the percentage crepancy found. The mean was 4.53% which corresponds to 2.24% in n or Values of ψ and C_D by this formula are tabulated below:

Batten spacing	ψ	C_D
24 in.	1.65	1.286
12 in.	2.25	1.190
6 in.	3.23	1.062
3 in.	4.32	0.790

The fact that C_D decreases with the spacing seems to indicate that even at these spacings the roughnesses are not entirely isolated, and that especially at the 3 in. spacing there is some wake interference. Also, the fact that none of the C_D's approach the 1.9 value given by Morris, throws some doubt on his 67.2 value.

As the mean discrepancies in velocity resulting from a given Q and S by the three methods are 2.05%, 2.18%, and 2.24%, none is outstanding. Because of its simplicity and familiarity Manning's would naturally be chosen, but it must be realized that the formula suggested here, based on Morris' formula, is the only one of the three which gives a definite value in terms of batten height and spacing.

Turning now to the runs with Froude number 0.70 or more, there are only three spacings, as no runs were made with a 3 in. spacing at a steep enough slope to give critical or shooting flow. The average values of Manning's n and the average percentage discrepancy from the average for each spacing are as follows:

Batten spacing	Value of n	Average discrepancy
24 in.	0.0142	3.17%
12 in.	0.0173	6.24%
6 in.	0.0195	2.88%

The weighted average discrepancy was 4.72%.

The same runs were computed on the basis of Nikuradse's rough pipe formula with the following results:

Batten spacing	Average ϵ	Average discrepancy in f
24 in.	0.0092 ft	5.52%
12 in.	0.0279 ft	10.00%
6 in.	0.0524 ft	7.70%

The weighted average percentage discrepancy in f was 8.12% which corresponds to 3.98% error in n or V.

An attempt was made to develop a formula connecting Ψ and λ as was done for tranquil flow, but the scatter was so great that the result seemed worthless. The Nikuradse type formula has a slight edge over the Manning formula for shooting flow on the basis of these tests, but the discrepancies from both formulas are so large that the whole question remains in an unsatisfactory state. In spite of the experimental errors present in these tests, it is believed that they show the resistance to shooting flow in triangular channels to be definitely greater than for tranquil flow for corresponding conditions (i.e. at the same relative roughness for rough channel flow and the same Reynolds number for smooth channel flow). This is contrary to the results found for

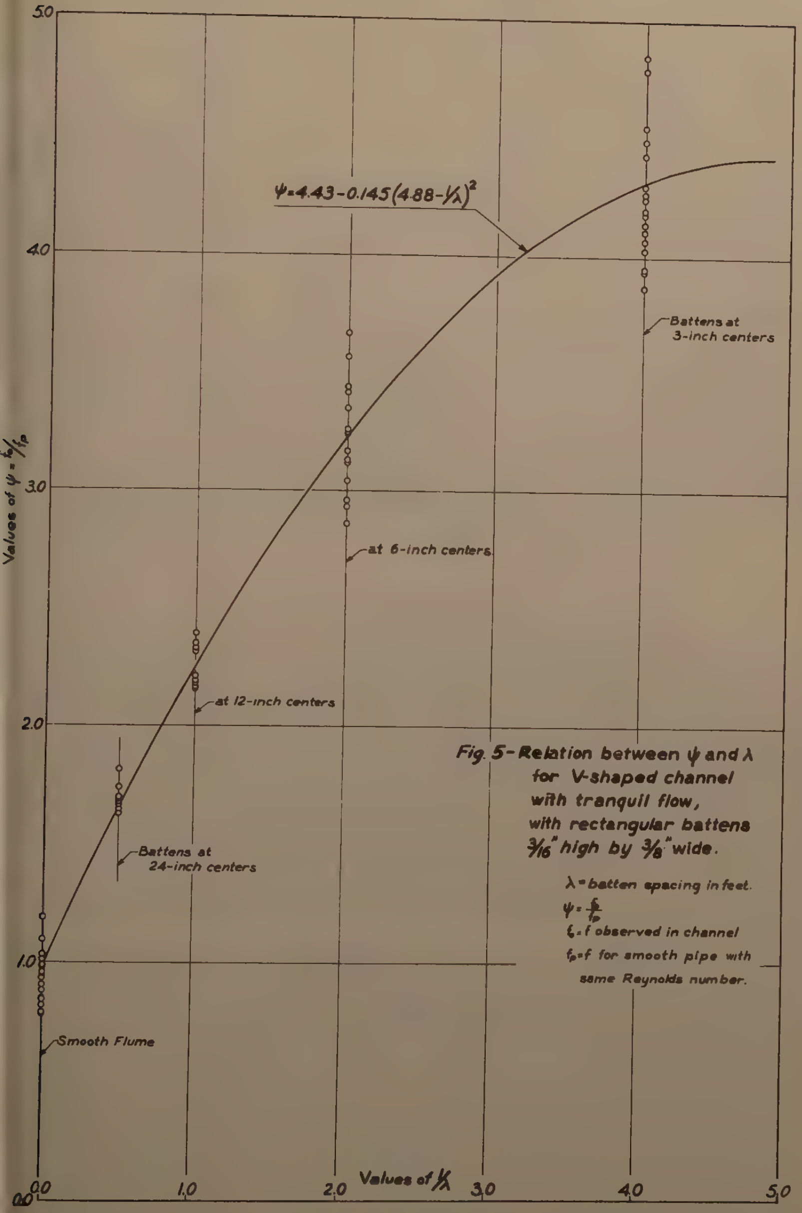


Fig. 5 – Relation between ψ and λ for V-shaped channel with tranquil flow, with rectangular battens 3/16" high by 3/8" wide.

rectangular channels by Powell.[8] (As has been pointed out elsewhere the terminology in that paper was incorrect as Table 1 (a) included both tranquil and shooting flow and (b) was ultra-rapid flow. Ultra-rapid flow was not obtained in the tests reported here. To obtain it in a triangular channel requires a much higher Froude number than in a rectangular channel.)

Increased resistance for flow at the higher Froude numbers can be explained on the following logical basis. It was recorded by Montana[18] that the "capillary rise" of water along the sides of a rectangular flume increased greatly when the velocity in the flume became supercritical. The "fillet" at the water's edge, normally very small, was apparently invaded by turbulence which so modified the balance of forces that the "fillet" enlarged to 3/4 in. on a side.

This phenomenon undoubtedly occurred in the present tests and in addition, it was noted that water from standing and transitory waves caused by roughness and turbulence "rode up" the sloping side and fell back with considerably decreased momentum. In spite of the dryness of the climate at the laboratory, the flume was kept wet for a distance above the normal shore line, sometimes 2 in. or more above it. High speed photographs show this condition and the retarded water in the waves. Clearly, this would add to the effective wetted perimeter, and cause an apparent increase in roughness if no additional wetted perimeter was taken into account. Possibly this effect could be evaluated.

Velocity Distribution

Pitot-tube velocity traverses were made for the smooth flume and for the different batten roughnesses. Despite the fact that more than one reading was seldom taken at a given point in the cross-section, the process was time-consuming. When two or three readings were taken at a fixed point, the variability of the results obtained showed that in order to establish a reliable value for the average, perhaps as many as twenty readings would have to be taken. For this reason the traverses, a sample of which is shown in Fig. 6, cannot be considered to show an accurate average velocity at each point. They do, however, provide a fairly reliable check on the discharge and value of α since the average of many observations is used in arriving at these values.

Of more general interest are pitot measurements made to obtain the variation of velocity perpendicular to the mid-point of the side of the flume. Close to the flume wall and in the middle of a long straight portion of the wetted perimeter, the variation of velocity ought to be free of any effect from the relatively distant water surface, or other segments of the cross-section boundary. If this relationship could be established in terms of roughness and flow parameters, the mutual effect of the different boundary segments could be studied by means of interaction diagrams or other suitable techniques, thus yielding a more general means of evaluating the effect of cross-sectional shape. Use of the hydraulic radius implicitly assumes either constant or no mutual effect of velocity gradients generated at the different boundary segments. While this causes little difficulty in comparing cross-sections that are not too dissimilar, results are known to be grossly in error when the cross-section is unusual, and such sections are customarily divided up into sub-areas and the hydraulic radius, average velocity, and discharge of each computed separately.

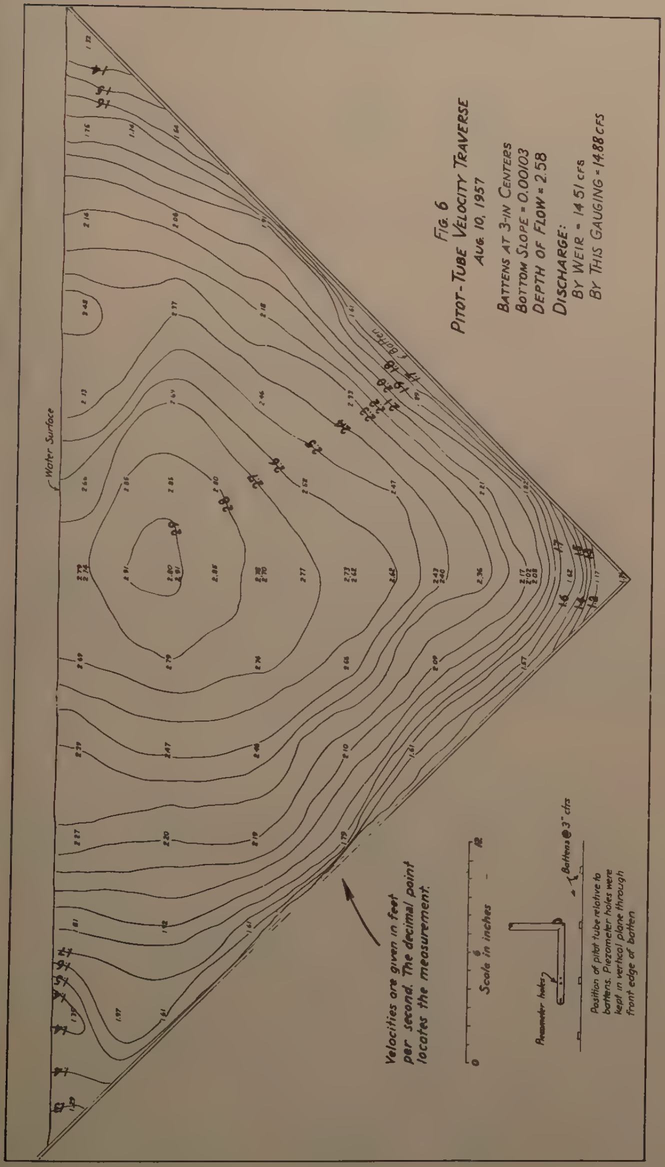


FIG. 6
PITOT-TUBE VELOCITY TRAVERSE
AUG. 10, 1957

The velocity determinations along the perpendicular to the mid-point of t wall were made at closely-spaced points near the boundary, though it was realized that the pitot indications might not be reliable when the tube was he too close to the side. Four or more readings were made at each location, b as can be seen from Fig. 7, the variability is so great as to suggest that twenty or more measurements would be required to establish the average value accurately. Logarithmic and semi-logarithmic plots of the data were made, and empirical relations of either type could be fitted fairly well. The logarithmic type being simpler, it was used for fitting the plots for the diffe ent roughnesses, as shown in Fig. 8. In each case the fit is fairly good near the wall, and diverges near the middle of the cross-section where the effect of the other boundaries becomes appreciable.

Secondary Currents

During the 1956 tests, it was noticed that floating debris thrown into the channel tended to become concentrated near the centerline as it moved down stream. This seemed to indicate the existence of secondary flow. Moeller-Hartmann(15) gives a complicated pattern of secondary currents deduced fr the velocity distribution in a cross-section. No instruments were available that would permit the relatively weak secondary currents to be measured in such turbulent flow, but the relative strength of the current near the surfac was roughly measured by the following scheme. A large number of small pieces of wood, approximately 3/8 in. in diameter by 3/4 in. long were soal in water long enough so that they barely floated. Then, for runs with differ roughnesses and very nearly the same depth, particles were released from rest, one by one, at the shore and their distance from shoreline noted after they had travelled 16, 32, 48 and 64 ft downstream from the point of releas The results are shown in Table II and are plotted in Fig. 9.

Table II

Date	Batten Spacing	Depth of Flow	Number of Values Averaged	Average off-shore distance in fe travelled while floating downstre			
				16 ft	32 ft	48 ft	64 ft
Aug. 19, 1957	6 in.	2.50	30-32	.91	1.58	2.03	2.28
Aug. 23, 1957	12 in.	2.53	37-40	1.11	1.67	2.07	2.35
Aug. 26, 1957	24 in.	2.35	27-28	.84	1.02	1.31	1.61
Aug. 30, 1957	No battens	2.52	31	.79	1.18	1.44	1.73

On the basis of this evidence, admittedly scanty, it would seem that for turbulent flow in the triangular cross-section there is a large weak secon spiral that is directed inward at the top, from the shoreline to the centerli (Although the floating-particle measurements were all made from the nort bank, there would presumably be another spiral, symmetrical about the centerline, as suggested by the 1956 observation.) There is a temptation

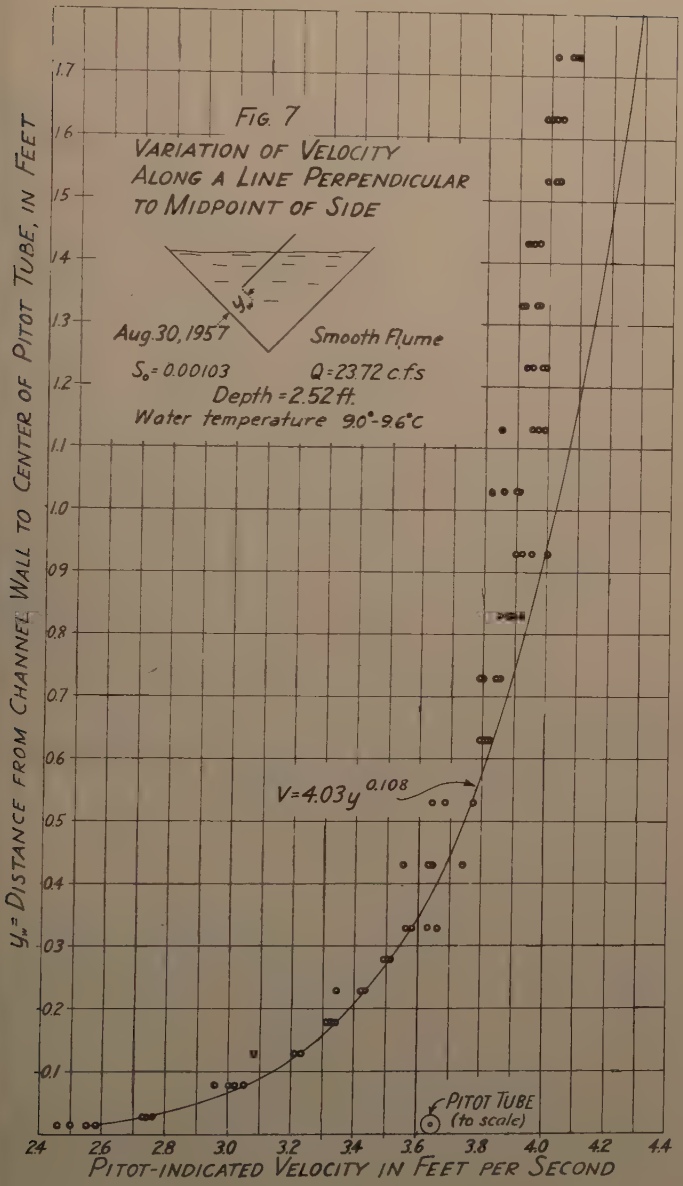


FIG. 7
VARIATION OF VELOCITY ALONG A LINE PERPENDICULAR TO MIDPOINT OF SIDE
Aug. 30, 1957
Smooth Flume
S_o = 0.00103
Q = 23.72 c.f.s
Depth = 2.52 ft.
Water temperature 9.0°-9.6°C
$V = 4.03y^{0.108}$
PITOT TUBE (to scale)
y_w = DISTANCE FROM CHANNEL WALL TO CENTER OF PITOT TUBE, IN FEET
PITOT-INDICATED VELOCITY IN FEET PER SECOND
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0
4.2
4.4
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
1.6
1.7

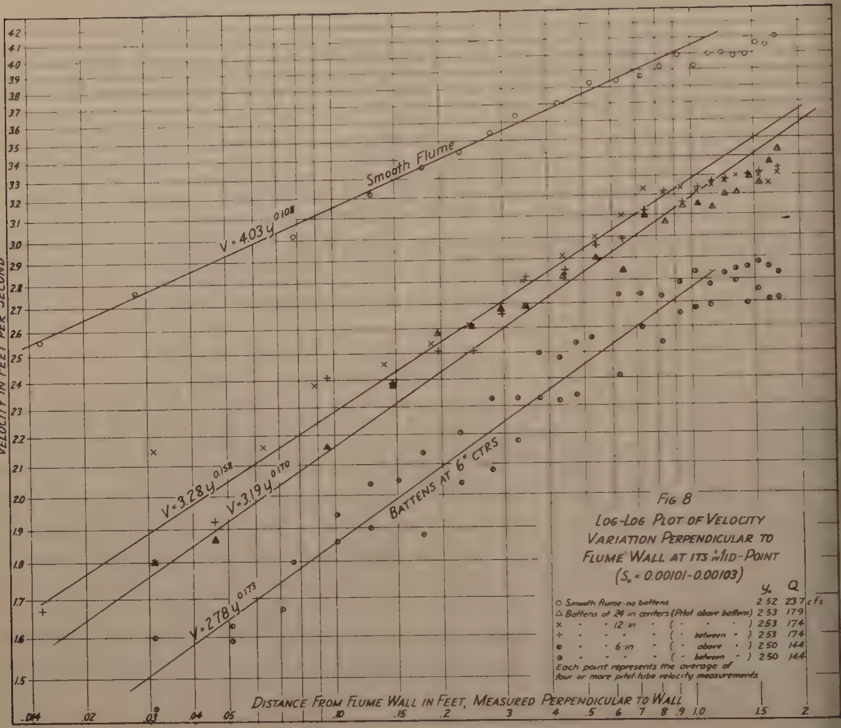


Fig 8
Log-Log Plot of Velocity Variation Perpendicular to Flume Wall at its Mid-Point (S_o = 0.00101-0.00103)

conclude that the spiral is strongest for the 12 in. batten spacing, but more data should be gathered before such a conclusion can be regarded as established.

Backwater Curves

Although the long variable-slope flume could well be used to investigate some of the cases of the backwater curves, the objective of the present stud was the evaluation of losses in uniform flow, and the only data that might be useful to test backwater curve theory were those incidentally obtained while establishing uniform flow. After the practice of plotting the profiles at the time observations were made was started very little data of any possible value for backwater comparisons was taken, but before that some interestin profiles were obtained which seemed to justify analysis as backwater curve These computations were made by Mr. K. P. Singh[20] as a master's thesis project in the Department of Mechanics and Hydraulics of the State Univers of Iowa. Fig. 10, reproduced from Mr. Singh's thesis, shows a typical com parison of observed data and theoretical backwater curves. In general, Mr Singh found that the assumption ordinarily made in computing backwater curves, that the rate of friction loss is the same as it would be for uniform flow at a given depth, gives very close agreement if velocity-head changes are correctly evaluated. He compared a number of recently-proposed back water curve methods, and prepared a new table of functions that permits

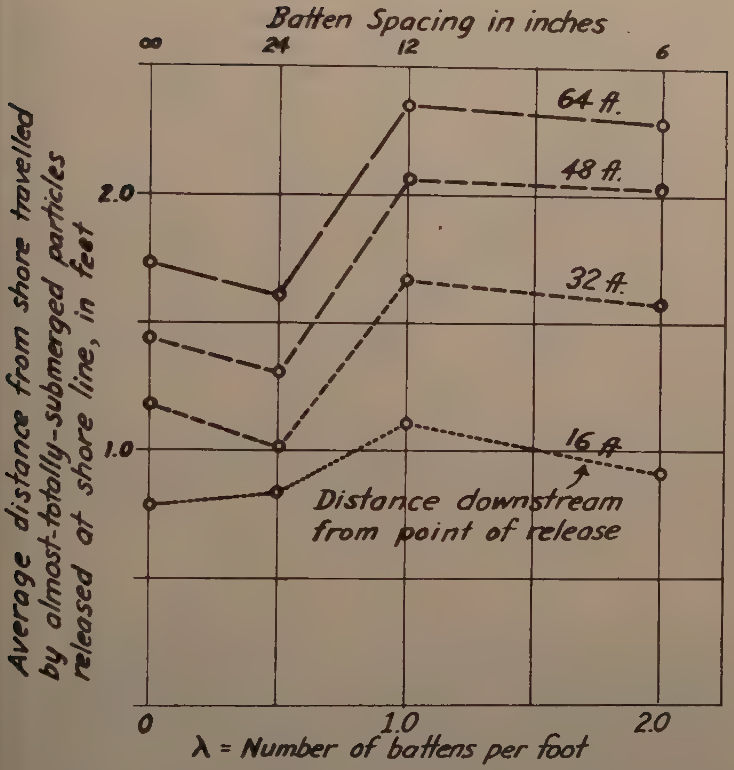


Fig. 9. Floating-Particle Test of Strength of Secondary Current

d computation of backwater curves in channels of triangular cross-
.on.

CONCLUSIONS

nis paper has reported on 156 runs on a long adjustable-slope 90° tri-
lar flume. The length of the flume was found to be sufficient for
letermination of normal depths up to approximately 2.5 ft. Including
uns classed as critical flow along with those classed as shooting,
esults may be summarized as follows:

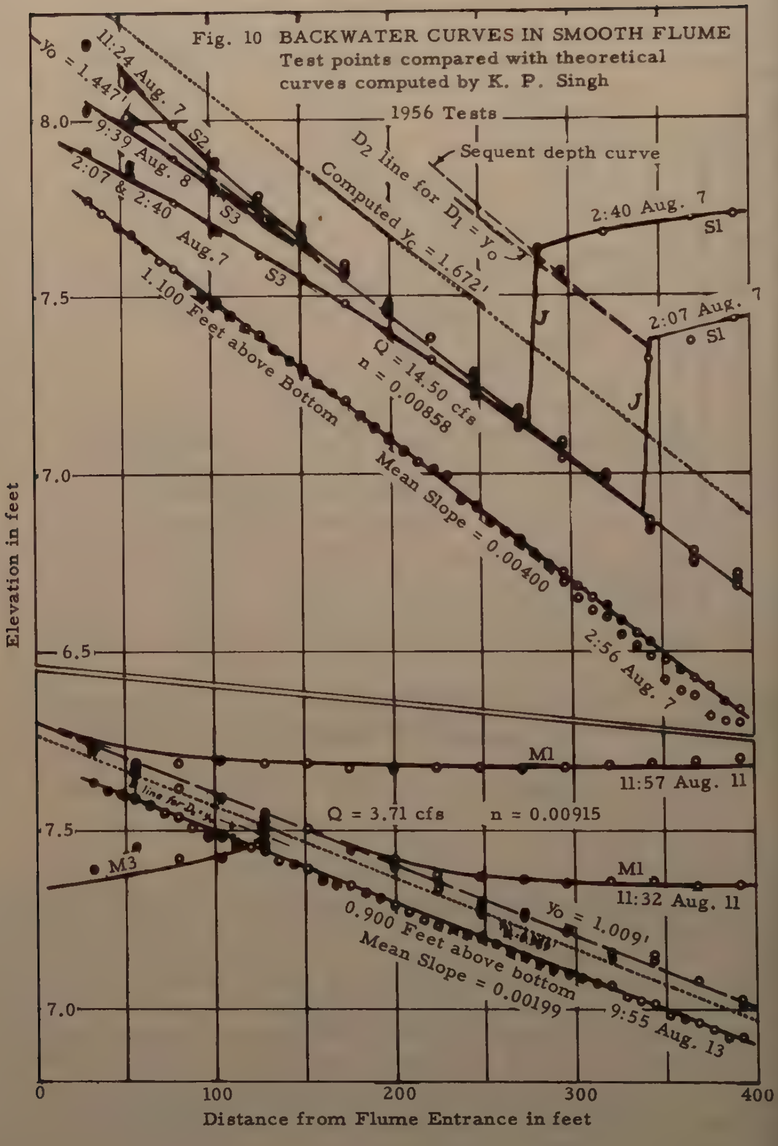


Fig. 10 BACKWATER CURVES IN SMOOTH FLUME
Test points compared with theoretical
curves computed by K. P. Singh
1956 Tests
Sequent depth curve
D_2 line for $D_1 = y_o$
Computed $y_c = 1.672'$
$y_o = 1.447'$
11:24 Aug. 7
S2
9:39 Aug. 8
S3
2:07 & 2:40
Aug. 7
S3
1.100 Feet above Bottom
Q = 14.50 cfs
n = 0.00858
Mean Slope = 0.00400
2:40 Aug. 7
S1
J
2:07 Aug. 7
S1
J
2:56 Aug. 7
M1
11:57 Aug. 11
Q = 3.71 cfs
n = 0.00915
M3
M1
11:32 Aug. 11
$y_o = 1.009'$
0.900 Feet above bottom
Mean Slope = 0.00199
9:55 Aug. 13
8.0
7.5
7.0
6.5
7.5
7.0
0
100
200
300
400
Elevation in feet
Distance from Flume Entrance in feet

Item	Smooth	Rough, batten spacing:- 24 in.	12 in.	6 in.	3 in.	Total
ber of runs, tranquil	23	9	9	16	18	75
ber of runs, shooting	33	16	24	8	0	81
ber of runs, total	56	25	33	24	18	156
ing's n, tranquil	.0092	.0128	.0150	.0180	.0210	
ing's n, shooting	.0094	.0142	.0173	.0195	--	
of f_o to f_p, tranquil	0.93	1.65	2.25	3.23	4.32	
of f_o to f_p, shooting	1.15	--	--	--	--	
ness ϵ in ft, tranquil		.0046	.0126	.0343	.0694	
ness ϵ in ft, shooting		.0092	.0279	.0524	--	

ese tests indicate that for this shape of channel the resistance is greater the flow is critical or shooting than when it is tranquil. For the case of uil flow with battens 3/16 in. high by 3/8 in. wide spaced λ ft apart cen- center, the velocity was given with a mean discrepancy of 2.24% by the ula

$$f = f_p \left[4.43 - 0.145\,(4.88 - 1/\lambda)^2\right]$$

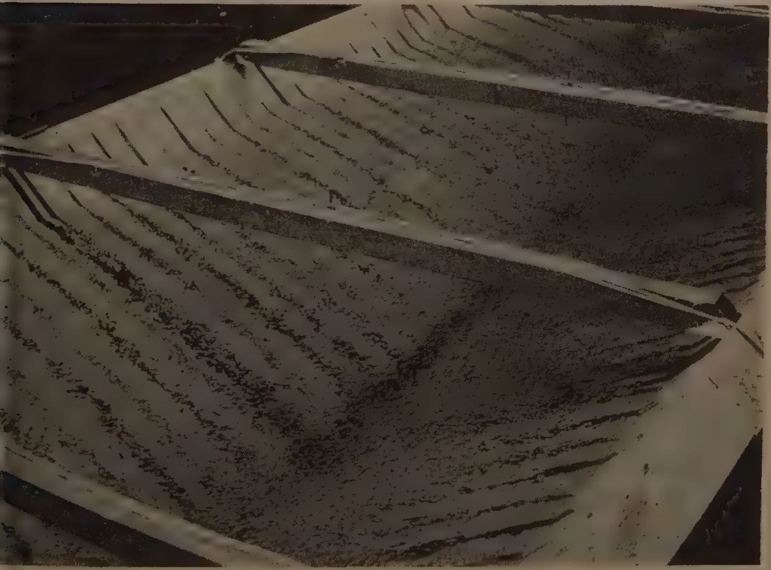


Fig. 11. Tranquil flow, battens at 3-inch centers.

where f_p is the f for flow in a smooth pipe with D = 4 R and the same Reynolds number, as given by Prandtl's formula. This also applies (but with a somewhat larger mean discrepancy) to flow in the smooth channel (by makin λ infinite). The authors were unable to derive a similar formula for critica or shooting flow.

A surprising result is that on the average these tests showed a smaller loss for tranquil flow in the smooth channel than for the corresponding smoo pipe. Also contrary to the presuppositions of the authors, it was found that Manning's formula gave about as good results as a logarithmic one, both fo the smooth and rough channels.

Pitot-tube traverses indicated discharges that agreed fairly well with the weir measurements. The values of α varied from 1.03 to 1.09.

Study of the velocity distribution for different roughnesses showed that the variation of velocity near the wall could be represented by exponential type formulas.

The existence of two secondary currents with surface direction from sho to centerline was inferred from measurements of the motion of nearly-submerged floats, and the strength of these currents compared for the different roughnesses.

The ordinary assumptions made in computing backwater curves were fou to be justified, to the extent permitted by comparisons with the backwater curves incidentally obtained in getting the normal depths.

ACKNOWLEDGMENTS

National Science Foundation support and U.S.G.S. and institutional coope tion have been acknowledged in the introduction. It remains to list the individuals who have assisted in carrying these tests to completion, sometimes under most difficult circumstances: Richard Tice, H. J. Koloseus, H. M. Fitch, J. C. I. Dooge, W. Donahue, Preston Farrell, Leigh Nason, Cornelius Shih, T. A. Strelkoff, Richard Warnock, Bertrand Barnérias, P. K. Mohanty Hau Wong Ho, G. Aron, Ram Garde, Peter Ellsworth, Peter Landweber, D. Donnan, B. Lettau, Palmer Sekora, Barton Bickel, and James B. Posey. Emory W. Lane and Ralph Parshall contributed valuable advice during the planning and conduct of the tests, and R. E. Glover supplied useful data obtained while making special diffusion tests in the long flume.

Notation

A - a constant in Bazin's formula; total area of cross-section in ft^2.

a - a constant in Bazin's formula.

b - length of weir crest: a constant in Bazin's formula.

C - Chezy coefficient = $V/\sqrt{R\,S}$.

C_D - coefficient of drag.

D - diameter of pipe in ft.

dA - partial area of cross-section.

Fig. 12. Rapid flow, battens at 12-inch centers. $^1/_{500}$ - second exposure shows ride-up on shore. Height of wetting not visible on far shore, but can be seen on near shore at bottom of picture.

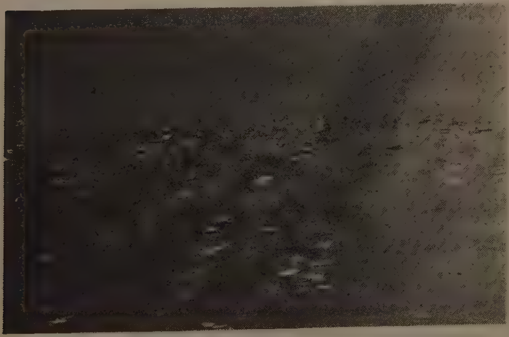


Fig. 13. Close-up of rapid flow in smooth flume showing ride-up and height of wetting.

f - constant in Weisbach formula = $8\ g\ R\ S/V^2$.

f_o - f as computed from observations on flume.

f_p - f for smooth pipe under comparable conditions.

g - acceleration of gravity in ft/sec^2.

h - head on weir in ft.

K - coefficient in weir formula.

n - coefficient in Manning formula.

P - height of weir crest above floor.

Q - discharge in cfs.

R - hydraulic radius in ft.

$\underline{R}$ - Reynolds number for weir = $g^{0.5}\ h^{1.5}/\upsilon$.

IR - Reynolds number for channel flow = $4\ R\ V/\upsilon$.

r - V_a/V_b.

S - slope of energy gradient.

V - mean velocity in cross-section.

V_a - mean velocity of approach above weir crest.

V_b - mean velocity of approach below weir crest.

v - velocity in area dA.

$\underline{W}$ - Weber number for weir = $\rho g h^2/\sigma$.

y_n - Depth for uniform flow.

α - velocity distribution factor = $\dfrac{\int v^3\ dA}{V^3\ A}$

γ - a constant in Bazin's formula.

ϵ - equivalent Nikuradse roughness in ft.

λ - a constant in Gaukler's formula; batten spacing in ft.

υ - kinematic viscosity in ft^2/sec.

ρ - density in slugs/ft^2.

σ - surface tension in lb/ft.

Ψ - f_o/f_p.

REFERENCES

1. Darcy, H. and Henry Emile Bazin, "Recherches Hydrauliques," Mémoire présentés par divers savants, Science mathematiques et physiques, Serie 2, Vol. 19, Paris, 1865.

Manning, Robert, "On the Flow of Water in Open Channels and Pipes," Trans. Institute of Civil Engineers, Ireland, Vol. 20, 1891, pp. 161-207.

Schoder, E. W. and K. B. Turner, "Precise Weir Measurements," Trans. ASCE, Vol. 93 (1929) pp 999-1190.

Keulegan, Garbis H., "Laws of Turbulent Flow in Open Channels," Research Paper 1151, National Bureau of Standards, Vol. 21, Dec. 1938, pp 707-741; and "Equation of Motion for Steady Mean Flow of Water in Open Channels," Research Paper 1488, National Bureau of Standards, Vol. 29, July 1942, pp 97-111.

Rogers, Thomas De F., "Friction in Hydraulic Models," Civil Engineering, Vol. 9 (1939),p 367.

Lenz, Arno T., "Viscosity and Surface Tension Effects on V-Notch Weir Coefficients," Trans. ASCE, Vol. 108 (1943), pp 759-802.

Johnson, J. W. "Rectangular Artificial Roughness in Open Channels," Trans. Am. Geophysical Union, Hydrology Section, 1944.

Powell, Ralph W., "Flow in a Channel of Definite Roughness," Trans. ASCE, Vol. 111 (1946), pp 531-566.

Kirschmer, Otto, "Pertes de charge dans les conduites forceés et les canaux decouverts," Revue Generale de l'Hydraulique, Paris, Vol. 15, No. 51 (May-June 1949), pp 115-138.

Powell, Ralph W., "Resistance to Flow in Smooth Channels," Trans. Am. Geophysical Union, Vol. 30 (Dec. 1949), pp 875-878.

Powell, Ralph W., "Resistance to Flow in Rough Channels," Trans. Am. Geophysical Union, Vol. 31 (Aug. 1950), pp 575-582; and Vol. 32, No. 4, pp 607-613.

Robinson, A. R. and M. L. Albertson, "Artificial Roughness Standard for Open Channels," Trans. Am. Geophysical Union, Vol. 33 (1952), pp 881-888.

Owen, W. M., "Correlation Between Pipe Flow and Uniform Flow in Triangular Open Channel," Trans. Am. Geophysical Union, Vol. 34 (1953), pp 213-219.

Morris, Henry M., Jr., "Flow in Rough Conduits," Trans. ASCE, Vol. 120 (1955), pp 373-410.

Moeller-Hartmann, Walter, "Abfluss in offenen Dreiecksgerinnen," Mitteilungen der Hannoverschen Versuchsanstalt für Grundbau und Wasserbau, 1957, pp 16-85.

Powell, R. W. and C. J. Posey, "Tests on the Flow of Water in a Smooth V-shaped Flume," Report No. 21 of the Rocky Mountain Hydraulic Laboratory, Allenspark, Colorado, June 1957, p 6.

Powell, Ralph W., Discussion of "Discharge Characteristics of Rectangular Thin-Plate Weirs," Journal of the Hydraulics Division, ASCE, Vol. 84, HY 3, June 1958, pp 1690-23-27.

18. Montaña, Jaime M., "Experimental Study of the Free Over-Fall as a Function of the Froude Number," Unpublished thesis for Master of Science degree, State University of Iowa, 1945.

19. Chintu Lai, "Determination of Slope From Empirical Data," Unpublished thesis for Master of Science degree, State University of Iowa, 1957.

20. Singh, Krishan Piara, "Study of Backwater Curves in a Triangular Channel," Unpublished thesis for Master of Science degree, State University of Iowa, 1958.

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

MODEL-PROTOTYPE STUDY OF A PLUMBING DRAINAGE SYSTEM[a]

D. A. Gyorog,[1] F. M. Dawson[2] M. ASCE and E. C. Lundquist[3]

SUMMARY

This paper describes how dimensionless groups of variables may be used xpress the performance of a plumbing drainage system. It is also demon-ted that data obtained from tests on a scale model can be correlated with e from the prototype in terms of the dimensionless parameters developed.

SYNOPSIS

he results of the study described in this paper can be grouped into two gories. First, dimensionless analysis was used to reduce the number of meters required to describe the performance of a plumbing drainage em. The data obtained from experiments run on test drainage systems e put into the form of dimensionless parameters and plots were made to rmine the functional relationship existing between these parameters. nd, studies were made to determine whether the performance of a full- drainage system could be predicted from the performance curves plotted a geometrically similar scale model.

wo geometrically similar drainage systems were constructed; one was a size drainage system; the other was a half-size model of the same sys- The performance curves obtained from the tests on these two systems cated that the performance of the full-size system could have been pre-ed on the basis of observations made on the model. However, it should be

Discussion open until October 1, 1959. To extend the closing date one month, a ritten request must be filed with the Executive Secretary, ASCE. Paper 2019 is rt of the copyrighted Journal of the Hydraulics Division, Proceedings of the merican Society of Civil Engineers, Vol. 85, No. HY 5, May, 1959.

he research leading to this paper was performed by D. A. Gyorog on a llowship at the State Univ. of Iowa sponsored by the Crane Co.

str. in Mech. Eng., Univ. of Wisconsin, Madison, Wis.
ean, College of Eng., State Univ. of Iowa, Iowa City, Iowa.
rof. of Mech. Eng., State Univ. of Iowa, Iowa City, Iowa.

pointed out that only steady flow conditions were considered in this study.

A numerical illustration is presented at the end of this study which shows how the pressure at a typical point in a proposed system can be computed from data obtained from a scale model of the system.

Analysis of the Problem

A plumbing drainage system is composed of piping which carries water ar entrained foreign matter away from plumbing fixtures in a building. All plumbing fixtures are equipped with traps which prevent odors and filth in th drainage system from flowing back into the open portion of the fixture. Most traps are only a few inches in depth, and pressures in the drainage system must not deviate positively or negatively from atmospheric pressure by amounts greater than the pressure head in the traps. If the pressure deviations are greater than this, the traps may be siphoned or blown out, thus des troying their effectiveness as barriers against transmission of odors. Vent pipes opening to the atmosphere at the roof of the building and connected to the drainage system are used to prevent large pressure deviations from occurring in drainage systems. If the vent pipes are of the proper size and ar connected to the drainage system at the proper points, the pressure through out the drainage system will remain nearly atmospheric even when water from the fixtures is being discharged.

Laboratory experiments on full-size drainage systems are difficult and expensive to conduct because of their large size. It would be advantageous, therefore, if measurements made on a scale model could be used to predict flow and pressure conditions in a full-size system.

In the research to be described two simple drainage systems were constructed; One a full-size system, or prototype, and the other a half-size geometrically similar scale model of the full-size system. Figure 1 shows the geometrical design of both the prototype and the model drainage systems employed in this study.

The prototype system employed a 4 inch diameter vent stack, and the model a 2 inch diameter stack. Transparent plastic pipe was used so charac teristics of the flow could be observed. For each experimental run, pressures at the various points indicated in Fig. 1 were measured by manometer connected to pressure taps on the vent stack. A constant pressure was main tained at A during a given test run through control of movement of the invert air tank. Observations were simultaneously made to obtain sufficient data to calculate the air and water velocities through the system. All measurements and observations were made only after steady flow conditions had been attained.

The drainage system was defined as that portion of the apparatus bounded by points A, B and C as shown in Fig. 1. Point A was designated as the entrance to the vent stack of the system. In a real system this point would be open to the atmosphere, but for reasons to be explained later, the flow of air into or out of the experimental system, and therefore the pressure at point A was controlled by the inverted air tank shown in Fig. 1. The horizontal pipe through point B was used to admit water into the drainage system to simulate the discharge from a typical plumbing fixture.

During the experiments the pressures existing at typical points in the sys tem (such as D, E or F) for various air and water flow rates were studied closely. Under dynamically similar flow conditions in the model and prototyp

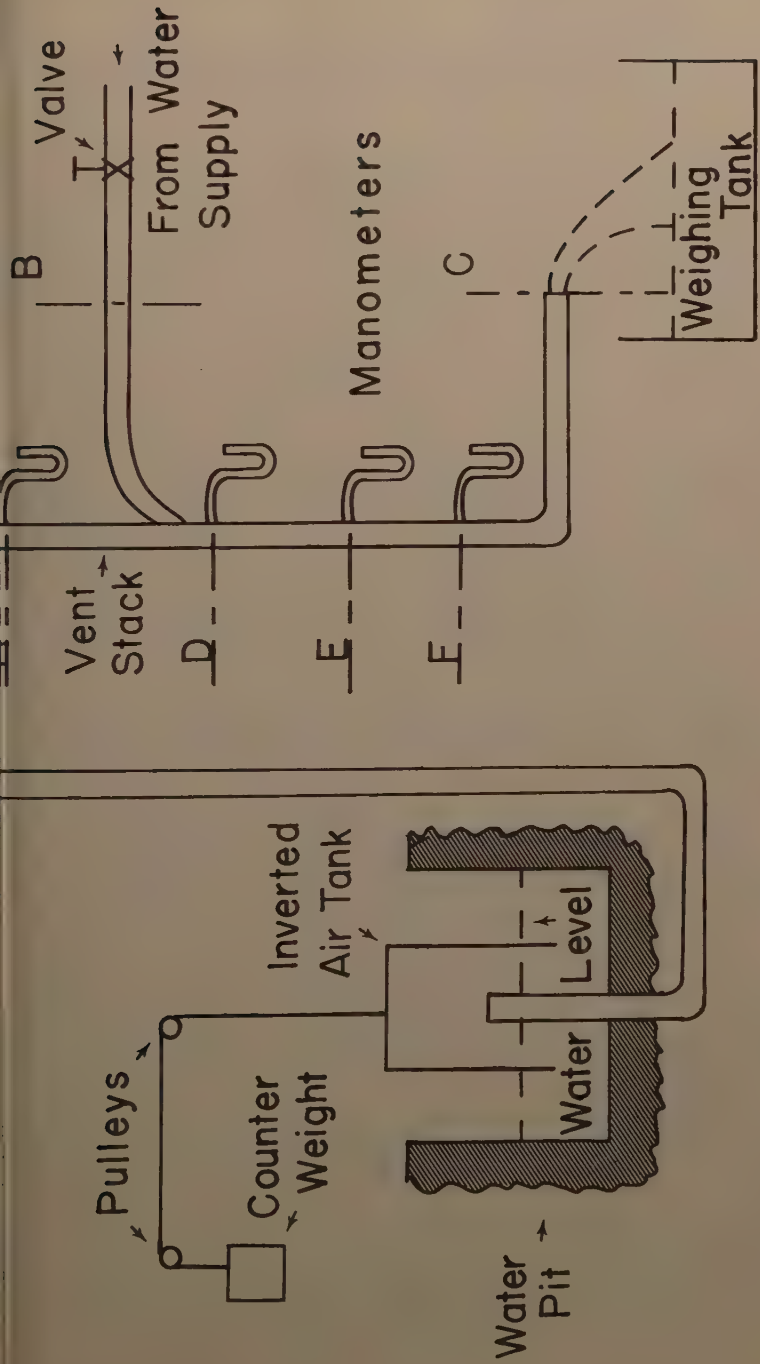


FIG. I EXPERIMENTAL APPARATUS

systems, it was found that measurements made on both systems could be co related using dimensionless parameters obtained from a combination of the variables affecting the systems.

The dependent variables in this problem are:

P_A-P, the pressure difference between the entrance to the stack and a point in the stack at which it is wished to determine the pressure (such as a D, E, or F in Fig. 1), lb/ft^2

V_A, the air velocity in the plumbing stack, ft/sec

The independent variables that must be considered are:

V_w, the water velocity in the plumbing stack, ft/sec

D, the diameter of the stack, ft

g, the acceleration of gravity, ft/sec^2

γ_A, density of the air, lbs/ft^3

γ_w, density of the water, lbs/ft^3, and

P_A-P_C, the pressure difference between the entrance and exit of the plumbing stack (at points A and C in Fig. 1), lb/ft^2.

It will be noted that two dependent and six independent variables are involved. The theory of dimensionless analysis was employed to combine thes eight variables into five dimensionless parameters. Furthermore, if no var ables other than the eight defined above are pertinent in the definition of the problem, the physical relationship existing between variables can be expressed as a function of the five independent dimensionless parameters that can be obtained employing this theory. In general, the theory of dimensionless analysis is useful for reducing the number of parameters required to express the function defining the physical phenomenon. The actual form of the function must be obtained from experiment or from other known facts as sociated with the phenomenon.

For this experiment Eq. (1) shows the five dimensionless parameters tha can be formed from the eight variables. The expression, f_1, implies that a functional relationship exists between these five parameters.

$$\frac{g(P_A - P)}{\gamma_A V_A^2} = f_1 \left[\frac{V_A}{V_W}, \frac{V_W}{\sqrt{Dg}}, \frac{\gamma_A}{\gamma_W}, \frac{g(P_A - P_C)}{\gamma_W V_W^2} \right] \qquad (1$$

Dynamically similar flow conditions will exist in the prototype and the model if all the parameters in the above equation have identical values for the prototype and the model. Therefore, if the form of the function is determined experimentally with a model system, the results should be applicable to the prototype.

The function expressed by Eq. (1) may be further simplified. If water an air are the fluids to be employed in the model, as they are in the prototype, the ratio γ_A/γ_W will be virtually constant over the range of conditions like to be encountered in either the prototype or the model. This is true becaus the pressure and temperature variations that are apt to occur in either are not of sufficient magnitude to produce appreciable change in the density of the water or the air.

e relationship (Eq. 1) then becomes

$$\frac{g(P_A - P)}{\gamma_A V_A^2} = f_2 \left[\frac{V_A}{V_W}, \frac{V_W}{\sqrt{Dg}}, \frac{g(P_A - P_C)}{\gamma_W V_W^2} \right] \tag{2}$$

the viscosities of the two fluids had been admitted as independent varia- forms of the Reynolds' number would have appeared in the functional onship. However, the effects of variations of the Reynolds' numbers neglected in these experiments. Experience in other fluid flow problems ates that variations of pressure coefficients caused by small variations ynolds' number are usually not great unless a critical region is en- ered. In this study it is argued that the viscosity effect and hence the olds' number is of little importance unless the scale of the model is very .

edure and Apparatus

e model and prototype systems were arranged as shown in Fig. 1. Point s designated as the entrance to the plumbing stack. The pressure at this was controlled by the magnitude of the counterweight on the inverted air The rate of air flow into the vertical stack past point A was obtained by urement of the rate of change of displacement of the inverted air tank. tack exit at C was open to the atmosphere, and differences in pressure en the entrance and exit (P_A-P_C) were created by controlling the rate of ment of the inverted air tank.

e pressure at the entrance to a plumbing vent is ordinarily atmospheric, e pressure at the drainage exit may be something other than atmos- c, as determined by the pressure conditions existing in the sewer to the drainage system is connected. In this investigation the situation eversed for simplicity. The drainage end of the system (C in Fig. 1) pen to the atmosphere, and the pressure was varied at the stack en- e (A in Fig. 2). In this manner, the same effective pressure difference P_C) was obtained. In addition, it was more convenient to measure the f air flow through the system by measuring the rate of displacement of verted air tank placed over the pipe entrance as previously described. ter flow into the system was controlled by a valve upstream from B, e rate of water flow was measured with a weighing tank and a stop . Pressures at various points along the vertical stack were measured nanometers, as shown in Fig. 1.

e velocities V_A and V_W were taken as the air and water flow rates ctively in cubic feet per second divided by the cross sectional area of ack in square feet. These velocities, then, were only proportional to ue velocities since the drainpipe was never running completely filled ither fluid at any one time.

s

(2) contains four variables, and it is necessary to employ two families ves as shown in Figs. 2 and 3 to express the relationship between these les. In these figures the quantities $g(P_A-P_C)/\gamma_W V_W^2$ and $g(P_A-P)/\gamma_A V_A^2$

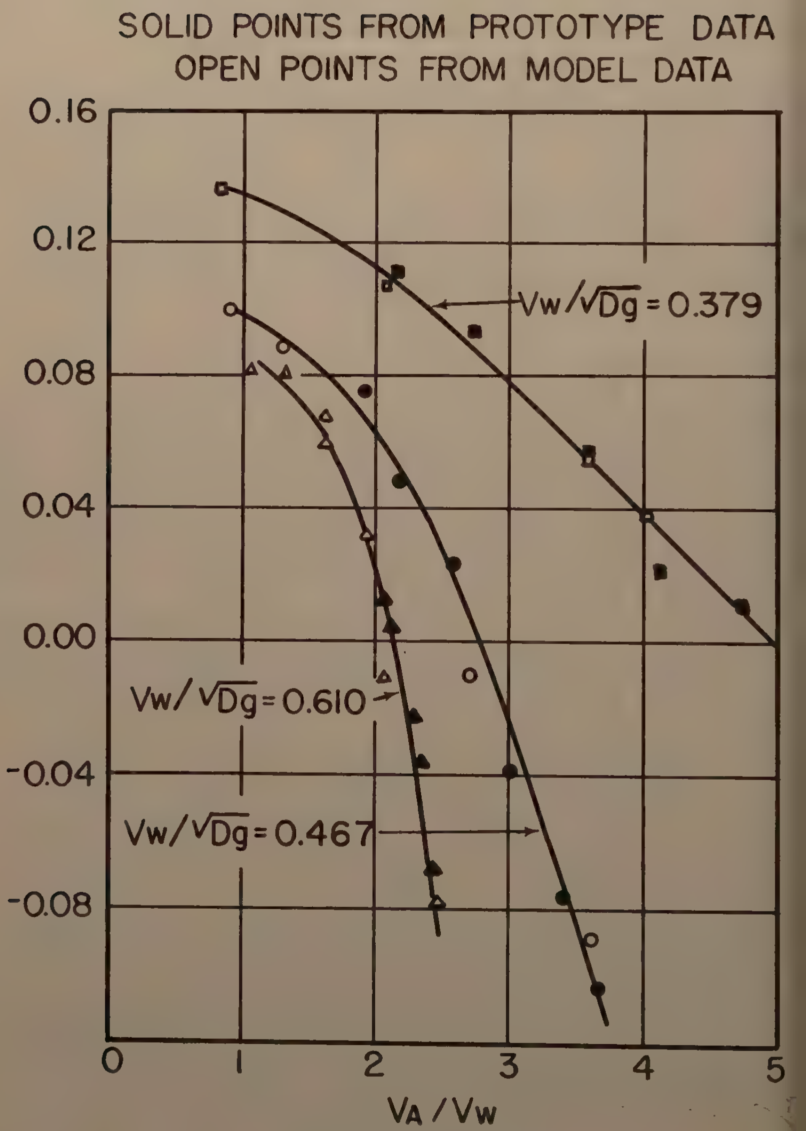


FIG. 2 EXPERIMENTAL RESULTS

plotted against V_A/V_W for constant values of $V_W/\sqrt{Dg}$. These curves applicable for only one location on the vent stack (point D, Fig. 1) at which ressure is P. Curves are not shown for other points along the stack, but were found to be of similar form.

igs. 2 and 3 show that test points obtained from both the prototype and nodel fall on common curves within the accuracy of the measurements. provides experimental verification of similitude in performance between rototype and the model.

ig. 3 may be used to predict the pressure in a similar drainage system at nt corresponding to that for which Fig. 3 is drawn (point D in Fig. 1). In ion, the required venting air flow rate may be computed if the water flow the pressure difference between the stack entrance and exit, and the eter of the stack are known. This presumes, of course, that the system hich the calculation is being carried out is geometrically similar to that to obtain test data.

n example will serve to clarify the use of these data. Suppose a prototype bing system with a stack diameter of 6" and geometrically similar to ised in this study were to be set up. The problem could be that of deter- ng the pressure at point D (the point for which the curves of Fig. 3 are n) and the air velocity required in the stack for a given water rate of and for a given pressure differential between the entrance and exit of the m (P_A-P_C). If the pressure at the stack entrance, P_A, is atmospheric, ressure difference would be determined by those factors governing the ure P_C. The flow conditions and discharge connections to the system stream from point C would govern the pressure P_C.

r this example, suppose the water flow rate is such that

$$V_W = 1.87 \text{ Ft./Sec.}$$

at

$$P_A - P_C = 0.433 \text{ Lb./Ft.}^2$$

it will follow that

$$\frac{g(P_A - P_C)}{\gamma_W V_W^2} = \frac{32.2(0.433)}{62.4(1.87)^2} = 0.064$$

$$\frac{V_W}{\sqrt{Dg}} = \frac{1.87}{\sqrt{0.5(32.2)}} = 0.467$$

shows that for these values

$$V_A/V_W = 2.0$$

Fig. 3 yields

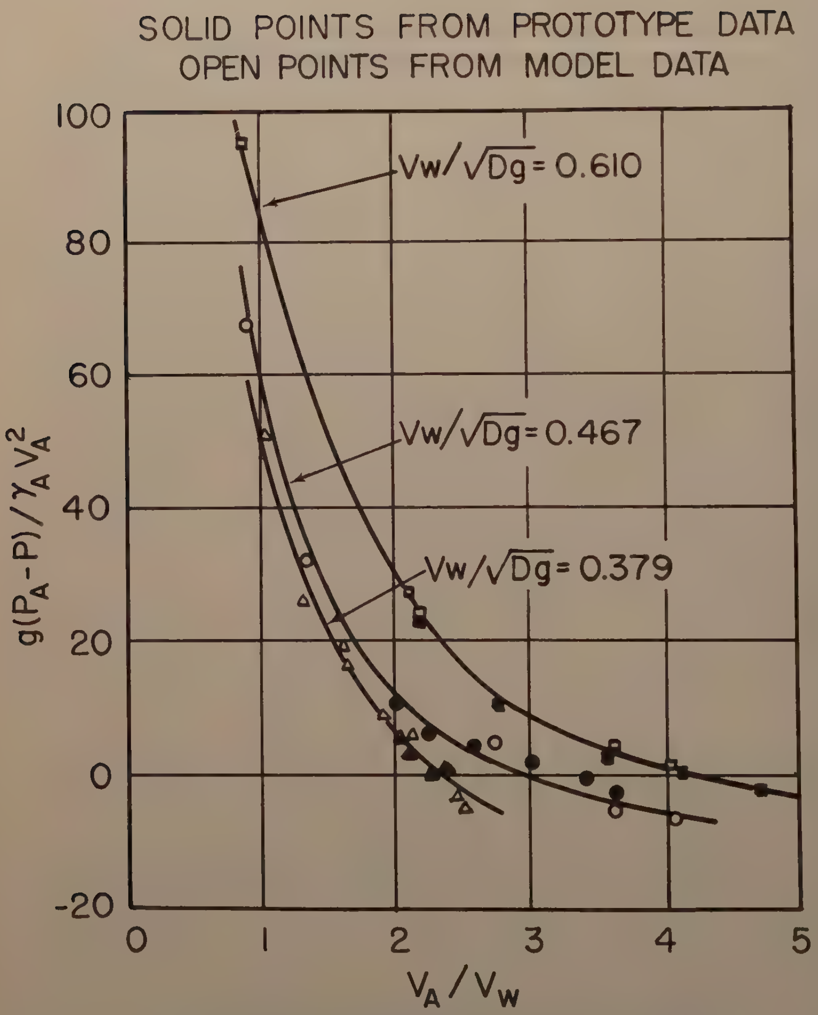


FIG. 3 EXPERIMENTAL RESULTS

$$\frac{g(P_A - P)}{\gamma_A V_A^2} = 12.0$$

lly, the following values are obtained

$$V_A = 2.0\ V_W = 2(1.87) = 3.74\ \text{Ft./Sec.}$$

$$P = \frac{12.0\ \gamma_A V_A^2}{g} = \frac{12.0(.075)(3.74)^2}{32.2} = 0.392\ \text{Lb./Ft.}^2$$

CONCLUSIONS

his investigation was carried out to determine the feasibility of employing ensionless groups of variables to reduce the number of parameters needed xpress the performance of a plumbing system. The technique was applied simple system, and the performance curves obtained are not intended to f value as specific design data. However, it has been demonstrated that from different size systems (so long as they are geometrically similar) be reduced to such a form that common curves will define the performance oth systems. Specifically, it has been shown that four parameters are suf- nt to define such performance so long as the difference in the size of the ems is not so large that possible variations due to Reynolds' number need onsidered.

Journal of the

HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

RESISTANCE PROPERTIES OF SEDIMENT-LADEN STREAMS

Vito A. Vanoni,[1] M. ASCE, and George N. Nomicos[2]

SYNOPSIS

:xperiments were made to determine the relative effects on the friction or of a flow of the suspended sediment load and the configurations which n on sand beds of streams. Results showed that the friction factor of a am carrying suspended sediment is less than a comparable one without ment. They also showed that the reduction of friction factor due to ıges in bed configuration is much larger than that due to suspended sedi- t.

INTRODUCTION

'he resistance to flow of clear water in a channel with fixed boundaries been studied extensively and can be predicted with a satisfactory degree ertainty. Values of friction factors to be used in the flow equations such ie Manning or Darcy-Weisbach formulas have been determined and are n in reference works on hydraulics as a function of the texture or rough- of the channel walls and the Reynolds number.

owever, when a stream has a movable bed, and sediment is being trans- ed, the problem of determining the resistance is much more complicated in the simple case of clear water flowing in a channel with fixed walls. rvations have shown that the friction factors for such sediment-bearing luvial streams vary over appreciable ranges. However, the problem has een studied sufficiently to enable hydraulic engineers to predict a friction or for given conditions.

Discussion open until October 1, 1959. To extend the closing date one month, a ritten request must be filed with the Executive Secretary, ASCE. Paper 2020 is ırt of the copyrighted Journal of the Hydraulics Division, Proceedings of the merican Society of Civil Engineers, Vol. 85, No. HY 5, May, 1959.

rof. of Hydrs., California Inst. of Technology, Pasadena, Calif.

ecialist—Scientific Research Engr., Republic Aviation Corp., Farming- ale, N. Y.

The fact that the friction factors for alluvial streams vary has been know for a long time. Despite this, the problem is not discussed at any length in the literature, and textbooks are particularly silent on this point. This leav all but the expert in this field with the erroneous impression that the frictio factor of streams is constant and can be determined once for all time. The lack of discussion in the literature of the resistance of alluvial streams ma be explained by the fact that the problem is not well understood, and informa tion on it is often conflicting. For instance, some evidence presented show that the friction factor of a stream increases as the sediment load increase while other evidence supports the opposite conclusion.

The variation of roughness of sediment-laden streams is caused by two distinct processes: (1) appearance of dunes and bars on the bed which may increase the roughness several fold because of the additional form resistan and (2) the damping effect of the suspended sediment on the turbulence in th stream tending to reduce the hydraulic roughness. Because of these two op posing processes, there has often been confusion about the effect of sedime load on roughness. Indeed, it is very difficult to separate the two effects b cause, for streams in which a significant amount of bed material is carried in suspension, the two processes are always acting together, but with differe relative effects. For example, in a sediment-laden stream flowing at high velocity over a flat bed of sand, the friction factor, or roughness coefficien will probably be lower than that for a clear flow over a fixed bed of compara ble grain roughness because of the damping effect of the suspended load. O the other hand, at a lower velocity with light suspended load, dunes may for and the net result will be an increase in the friction factor over that for a fixed flat bed of the same sand roughness.

This paper presents results of experiments in which an effort was made separate the two processes in order to determine qualitatively their relativ order of magnitude. To accomplish this, runs were made with suspended lo and a movable bed, and then with clear water but with no change in the bed configuration. This necessitated the development of a procedure for solidif ing a natural bed configuration in place so that the clear water could be mad to flow over exactly the same bed as the sediment-laden flow. A compariso of friction factors then showed directly the damping effect of the sediment separately from the effect of changing bed configuration. In the following pages a brief review of available information on the problem is given, and then the experiments are described and the results and conclusions derived from them are given.

Previous Studies of Resistance of Sediment-Laden Streams

Early Studies of the Resistance Problem

One serious obstacle to the understanding of the resistance of alluvial streams by the hydraulician of the latter part of the last century was the theory developed to explain the phenomenon. The reasoning back of the theo started out with the logical idea that the loss in energy head between any tw stations on a stream is merely the difference in elevation between the sta- tions. It was then argued that this drop, or head loss, was used up in two ways: (1) in overcoming the hydraulic friction of the flow, and (2) in trans- porting sediment. According to this concept, the more energy used in

ransporting sediment, the less was available to over come friction, and vice ersa. According to the Chezy formula, the velocity of a stream is propor-ional to the square root of the energy slope where the energy slope is merely ne loss of energy to overcome friction per unit length of stream. Now, if a urbid stream uses part of the total available energy to transport sediment, it vill have a smaller energy slope and hence a smaller velocity than it would ave if it carried no sediment. Thus, it was concluded that a clear stream vould have a larger energy slope and hence, according to the Chezy formula, vould flow faster than a comparable one carrying sediment.

In view of present knowledge of flow problems, it is clear that some of the deas upon which the above conclusion was based were incorrect. To begin vith, much of the energy to transport sediment comes from the turbulence vhich itself results from the expenditure of energy in overcoming the friction. Therefore, this energy of turbulence is no longer available to overcome fric-ion, and the fact that some of it is used to transport sediment should not nec-ssarily affect the dissipation of the main energy head. Also, apparently it vas not recognized that the friction, or roughness factor, of sediment-bearing treams varies widely with load and flow rate, and that because of this the nean velocity can vary even without a change in energy gradient or depth.

The energy concept held by the hydraulicians of the last century, and the eductions resulting from it, were so firmly established that they influenced ne thinking of many workers of that era, and some vestiges of the ideas are be found even in relatively recent literature. It appears from his early ublications that G. K. Gilbert[1,2] adhered firmly to this concept. This, no oubt, had an important influence on the thinking of his time because of the xcellent reputation he had amply earned in his active and productive career.

Latham, in 1886,[3] reported that observations of a storm sewer four feet n diameter in Croydon, England, over a period of years showed that the nean velocity for a given water depth was always greater when the water was lear than when it was turbid. He concluded that the retardation of the turbid ater was due to the load of sediment it was carrying. Hooker[2] p. 288, in ommenting on Latham's conclusions, noted that Gilbert also reached the ame conclusion. Hooker also went on to state, "The law of conservation of nergy will not admit any other decision in this matter, though the statement as sometimes been made that such a retardation of velocity does not exist." ne statement contradicting the belief that a stream at a given stage would ow faster when clear than when turbid, was made by McMath, and was in-uded in the paper by Hooker[2] footnote p. 289. McMath[4] was apparently ting some observations of the Mississippi River which contradicted the ac-epted views of the time.

In his classical flume experiments in sediment transportation, Gilbert[5] 229, expected to confirm the idea that a clear stream flows with less re-stance than a comparable loaded one. He selected pairs of runs from his periments which had the same depth and slope except that one of them car-ied sediment and the other did not. The bed in both cases was composed of pavement of sand grains which did not move. According to the then preva-nt theory, the mean velocity of the clear flow should exceed that of the aded flow. However, the opposite result was found in eleven of the sixteen irs of runs selected for the comparison. The depth measurements in these ns were apparently not as reliable as desired, and it was thought that this ay have introduced a systematic error into the result. Therefore, Gilbert lected 13 more sets of comparable runs, 11 of which showed higher velocity

for the clear flows than for the loaded flows. However, it was also noted that the reduction in mean velocity due to sediment transport was greater for small load than for large, for gentle slopes than for steep, for low velocity than for high, and for large depths than for small. As a result of these data, Gilbert was forced to admit that the theory was not supported and that there was probably some physical law which escaped his analysis.

A valuable contribution to the resistance problem was made by Buckley[6] in his work on the Nile. He made elaborate continuous observations of slope, mean velocity, and cross-sectional area at several stations on the river which showed an appreciable fluctuation of the coefficient of roughness. Typical values of these are shown in Table 1.

TABLE 1. Measurements at the Khannaq Nile Discharge Station by Buckley (6)

Date	River elev. meters	Cond. of flood	Flow rate $meter^3/sec$	Sed. conc. $grams/meter^3$ (parts per million)	Mean Vel. meter/se
Aug. 16, 1920	87.54	rising	7200	1900	1.48
Sept. 16, 1920	87.54	falling	6400	1000	1.32

Data applying to this station on both dates:

Cross-sectional area	4850 square meters
Slope	0.000072
Hydraulic radius	8.6 meters

These data show clearly that at the given stage the mean velocity of the river is larger on the rising than on the falling stage, while other channel characteristics remain unchanged. The reduction in mean velocity represents an increase in the Manning friction factor of 11 per cent. A further pertinent point to be noted is that the sediment concentration, like the velocity, is higher on the rising than on the falling flood.

Fig. 1 shows the measurements made by Buckley[6] in 1921 at Beleida Station on the Nile, which is about 28 miles upstream from Cairo and about 40 miles upstream from the Delta Barrage, or dam, which diverts water to the canals of the delta system. The variation in slope shown is due to the change in level of the water at the Delta Barrage required to feed the canals. The value of Kutter's roughness coefficient is seen to decrease appreciably as the discharge and mean velocity increase. For example, the following extreme values are obtained from Fig. 1:

	April 12, 1921	Sept. 8, 1921
Discharge, cu. meters/sec	500	6200
Kutter "n"	.063	.027
Hydraulic radius, meters	3.1	7.8

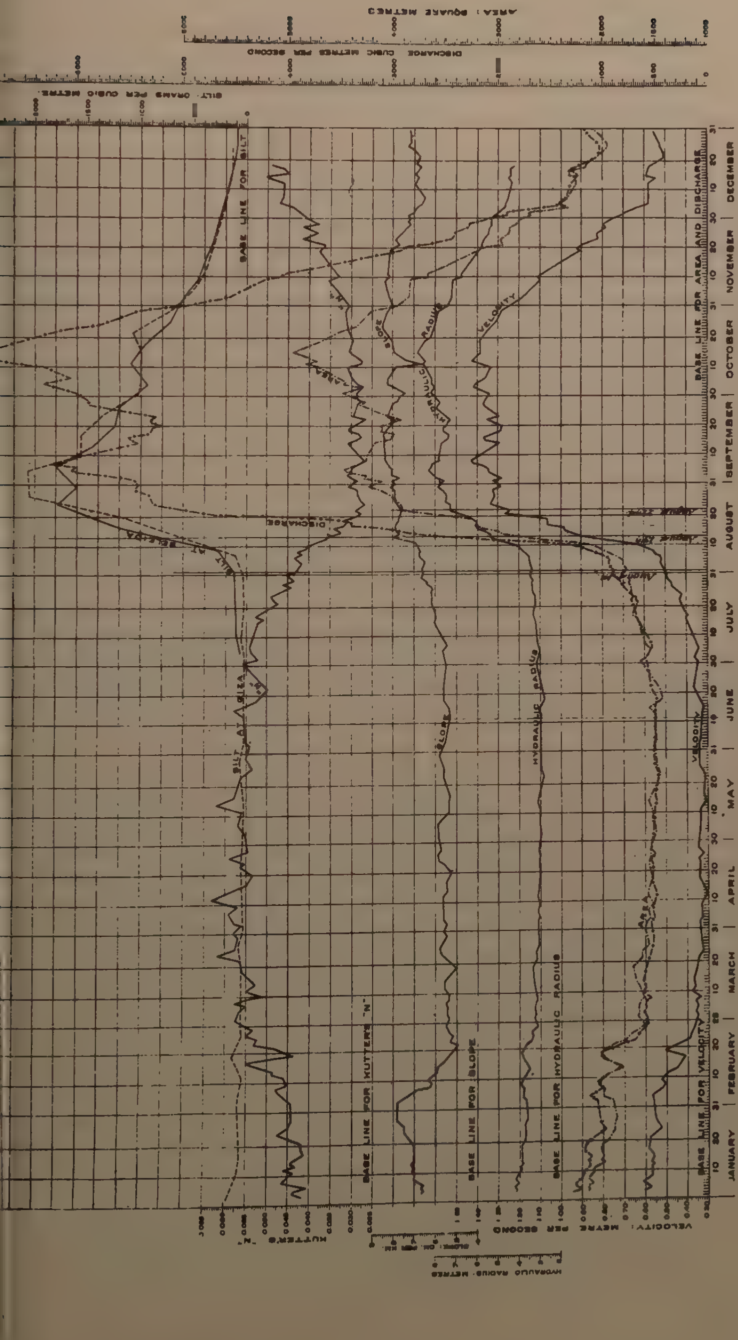


Fig. 1. Measurements on the Nile River at Beleida, reported by Buckley.(6)

Such great reduction in channel roughness during the flood must have been due primarily to smoothing out of dunes and bars on the river bed, with the damping effect of the suspended sediment being secondary, as will be seen from the laboratory experiments described below.

In discussing Buckley's paper(6) p. 231, Lacey pointed out that in the report of the Indus River Commission covering the period 1906 to 1910, it was stated that the velocity and discharge for a rising river were higher than for a falling river for the same stage, and the reverse was true for the water cross section of the stream.

Kantlack(6) p. 265, cited evidence which disagreed with that obtained by Buckley. He called attention to the result of research on rivers by Swiss and French engineers which showed that the stream velocity was decreased in every case as the sediment concentration increased. According to Kantlack, this had been demonstrated prior to 1870. He also stated that in his experience in the Punjab Canal system of India there was never an excess of flow during the time of high sediment load, as would be expected from Buckley's findings on the Nile and as was reported by other workers in India.

In the opening paragraph of his paper(6) Buckley also called attention to some laboratory work by Duponchel which showed that a sediment load tended to increase the mean velocity of flow.

Recent Studies of the Resistance Problem

Interest in the resistance problem in the past two or three decades has been stimulated by both laboratory and field studies of the sediment problem. One of the main objectives of research in sedimentation is to develop a theory which will enable one to predict the sediment load of a stream as a function of the water discharge and the stream channel characteristics. Once the discharge is given it is necessary to calculate the flow velocity and depth, which requires that one know the roughness coefficient of the channel. Thus the need for knowledge of resistance of streams arises naturally in dealing with sedimentation problems.

Experiments by Vanoni(7,8) in a flume with fixed, artificially roughened bed showed that a flow with suspended load had a smaller friction factor than a clear water flow of the same depth in the same channel. These experiments were made first with clear water and then with various small amounts of sand, so that the sediment load was controlled. Experiments by Ismail(9) in a rectangular pipe showed a decrease in friction factor with increase in load in some cases, but the opposite trend in others, thus duplicating the conflicting results reported by field observers. The apparent paradox was probably due to the fact that in some runs the bed became roughened with small dunes, making the roughness increase instead of decrease when sediment was added to the system.

Fig. 2 shows graphs of velocity profiles measured at the center of a flume for two flows of the same slope and depth, one with clear water and the other with a mean sediment concentration of 15.8 grams per liter of 0.1 mm sediment. It is clear that the sediment-laden flow which has the higher velocity has the lower friction factor. The right-hand graph, in which the relative distance up from the bed is plotted on a logarithmic scale, shows that the velocity profile follows the von Karman logarithmic law. For two-dimensional flow, profiles can be expressed by

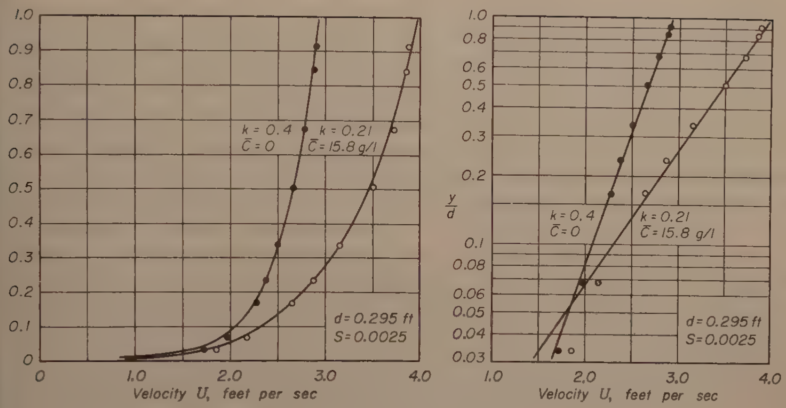


Fig. 2. Linear and semi-logarithmic graphs of velocity profiles in a flow 0.295 ft deep and 33.5 in. wide with clear water and with a heavy suspended load of 0.1 mm sand.

$$u = \bar{u} + \frac{1}{k}\sqrt{\frac{\tau_o}{\rho}} + \frac{2.3}{k}\sqrt{\frac{\tau_o}{\rho}}\,\log_{10}\frac{y}{d} \qquad (1)$$

ere u is the point velocity at a distance y up from the bed, $\bar{u}$ is the mean ocity at the profile, τ_o is the shear stress at the bed, ρ is the mass den- of the fluid, d is the flow depth, and k is the von Karman constant. From 1 one can write,

$$u_2 - u_1 = \frac{2.3}{k}\sqrt{\frac{\tau_o}{\rho}}\,\log_{10}\frac{y_2}{y_1} \qquad (2)$$

re u_1 and u_2 are the velocities at levels y_1 and y_2, respectively. Since can be calculated from the measured slope and depth, one can calculate k m a pair of points on the velocity profile by Eq. 2. The effect of the sus- ded sediment in reducing k, as indicated by Fig. 2, is a general one which urs at all times(8) in the laboratory as well as in the field.

Recent experiments by Brooks(10) demonstrated clearly how the friction or is affected by the dunes which develop on the sand bed of a stream. He ained friction factors for dune-covered beds which were as much as seven es as large as for flat sand beds of the same material. Studies by Ali and ertson,(11) and Barton and Lin(12) also showed clearly the importance of es on the channel roughness.

Recent studies of rivers have contributed appreciably to the understanding ne resistance problems. Leopold and Maddock(13) presented detailed ob- vations of streams which show that at a given discharge high sediment ls are associated with high mean velocities and low water depths. This ns that high loads are also accompanied by low friction factors, as was d on the Indus River system by Lacey(6) over 35 years ago.

Eden[14] made a study of Manning's roughness coefficient "n" as a function of river stage for the lower Mississippi River. He found that n varied from about 0.040 at low stages to about 0.030 or less at flood stages. Carey and Keller[15] have recently investigated the bed configuration of the lower Mississippi River by means of a sonic fathometer and found that sand waves were present at all times and that the largest waves were formed at the highest flow rates. They concluded that these waves must be a major factor in determining the friction coefficient of the river, although they were unable to measure it during their investigations. Carey and Keller are of the opinion that the loop in the rating curve of the River can be explained, at least partially, by a lag in the adjustment of the sand waves to the flow, i.e., during the rising flood the waves are growing and hence have a smaller resistance than they do at the same flow rate during the receding flood when the sand waves are diminishing.

Harrison[16] presented data on the Missouri River near Fort Randall which showed that the Manning roughness coefficient diminished with increase in discharge. The minimum roughness coefficient obtained by him was about equal to that which would be expected by a sand roughness equal to the grain size of the bed material, so it seemed reasonable to conclude that dunes existed on the bed at lower discharges.

Apparatus and Procedure

The equipment and techniques used in the experiments are essentially those described by Brooks[10] and therefore are described only briefly here. The experiments were carried out by Nomicos and are described in his thesis[17] as well as in a report to the Corps of Engineers, U.S. Army[18] who sponsored the research.

Flume

The flume used in these studies is 40 ft long, 10-1/2 inches wide, and 10 in. deep. It is equipped with an axial-flow pump with a variable speed drive, venturi meter, transparent section in the return pipe, and electric immersion heaters to control the temperature of the water. The open flume section is made of hot rolled steel, and its interior is painted with a smooth bitumastic paint. Along the top of the channel are mounted steel rails and a movable instrument carriage which can measure vertical elevations relative to the flume to the nearest 0.001 foot. The entire flume is mounted on a truss, the slope of which can be easily adjusted even during the course of a run. Although the channel itself is 10 in. deep, experiments were made with depths of flow less than 4 inches to avoid excessive wall effects on the velocity and shear distribution of the flow.

Depth and Slope

The elevation of the water surface and sand bed relative to the rails on the flume was read with the point gage. To get the bed elevation once the water surface had been measured, the flow was stopped and the water drained out. The bed was then leveled in reaches of 3 or 4 ft with a scraper carried on the instrument carriage rails. The elevation of this leveled surface was taken as the mean elevation of the bed. The water depth at any point was taken as the

erence between water surface and bed elevations and the local elevation of energy grade line was calculated by adding the velocity head based on an depth to the water surface elevation.

Profiles along the flume of the water surface, bed, mean depth, and energy de line were plotted for each run to determine if uniform flow obtained. ally several trial runs were required before uniform conditions were es- ished. The water surface and energy grade line were not always parallel he flume itself, even with uniform flow. Therefore, the actual slopes were ained by adding the slope of the flume and the slope of the water surface or rgy grade line relative to the flume.

ocity

The discharge was measured with a venturi meter with a throat diameter .00 in. placed in the return line, which was a 4-in. standard pipe. The an velocity was calculated as the discharge divided by the mean depth times flume width. Point velocities were measured with a 3/16 diam. Prandtl t-static tube and a water manometer reading to .001 ft.

iment Concentration

The mean sediment discharge concentration, $\overline{C}$, was determined by pling the flow in the suction well of the pump at the downstream end of flume. The sampler was a brass tube 0.302 in. in inside diam. with a -loop at the end so that when the tube was held vertically during sampling inlet faced upward into the flow which was downward.

Point concentrations, c, in the flume were determined from samples taken a 3/16 in. outside diam. brass tube flattened at the end so the tip opening .040 in. high by .217 in. wide. The tube was bent in a right angle much the common pitot tube, and was mounted on the instrument carriage with tip pointing upstream.

The velocity at the inlets to both samplers was kept the same as the local velocity by controlling the sampling rate. Each sample consisted of at t three one-liter bottles of the sediment-water mixture. The concentra- s were determined by filtering, drying, and weighing the sediment collected ach bottle.

perature

The water temperature in all runs was maintained at $25^{\circ} \pm 1^{\circ}$C. This was mplished with immersion heaters in the return pipe which could be regu- l manually to give power input ranging up to 4,000 watts in convenient s.

-wall corrections

ince the width-to-depth ratio of the flows in the experiments were small pared with natural streams, the effects of side-wall friction are relatively e important. To reduce the laboratory results to correspond to those h would obtain for much wider streams a correction is made. This fol- a method presented by Johnson(19) using a procedure developed by lks.(20,18) This yielded an hydraulic radius r_b, shear velocity U_{*b}, and ry-Weisbach friction factor f_b applying to the bed itself relatively free of

side-wall influence. The friction factor f_b is defined by the Darcy-Weisbach equation,

$$S = f_b \frac{1}{4r_b} \frac{V^2}{2g} \tag{3}$$

where S is the slope of the energy grade line, V is the mean flow velocity in the cross section, and g is the acceleration of gravity. To calculate the mean value, f, of the friction factor, one substitutes the hydraulic radius r of the section into Eq. (3) in place of r_b.

Sand Characteristics

Cumulative size frequency curves for the two sands used in these experiments are shown in Fig. 3, and the size characteristics are summarized in Table 2. These sands were prepared from white foundry sands with well-rounded grains. The material was almost entirely silica, so that the mean specific gravity of the grains was 2.65. The mean settling velocity, w, of the various sands used was determined from sedimentation diameters of sieve fractions measured by Vanoni[21] where the fractions were obtained with a next of sieves with openings of adjacent sieves in the ratio $\sqrt[4]{2}$. These sedimentation diameters are shown in Table 3.

Procedure for Experiments

A run with uniform flow was first established by the usual techniques with determinations of depth, water discharge, sediment discharge, and slope. After the final determination of the bed profile by the leveling process, the flow was again started in order to regenerate the bed configuration which was destroyed when the bed profile was taken. When the bed was reestablished, the flow was stopped and the water carefully drained off. The bed was then solidified by spraying with chemicals, as explained in detail in the following paragraph. After the chemicals had set, experiments with the stable bed were made, first using clear water and then with varying small amounts of sediment added to the system.

TABLE 2. Summary of Characteristics of Sand Used in Experiments

Sand No.	Experiments used in, Set No.	D_g Geom. mean diam. mm	g Geom. std. deviation	D_s Mean sed. diam. mm	w Mean fall velocity at 25°C. ft/sec
6	I, II, II	0.091	1.16	0.105	0.031
7	IV	0.148	1.16	0.161	0.062

Stabilization of sand bed

The method of stabilizing the sand bed was developed by tests of samples in trays and in actual flume tests. The most satisfactory method involved the use of sodium aluminate and sodium silicate as stabilizing chemicals. The

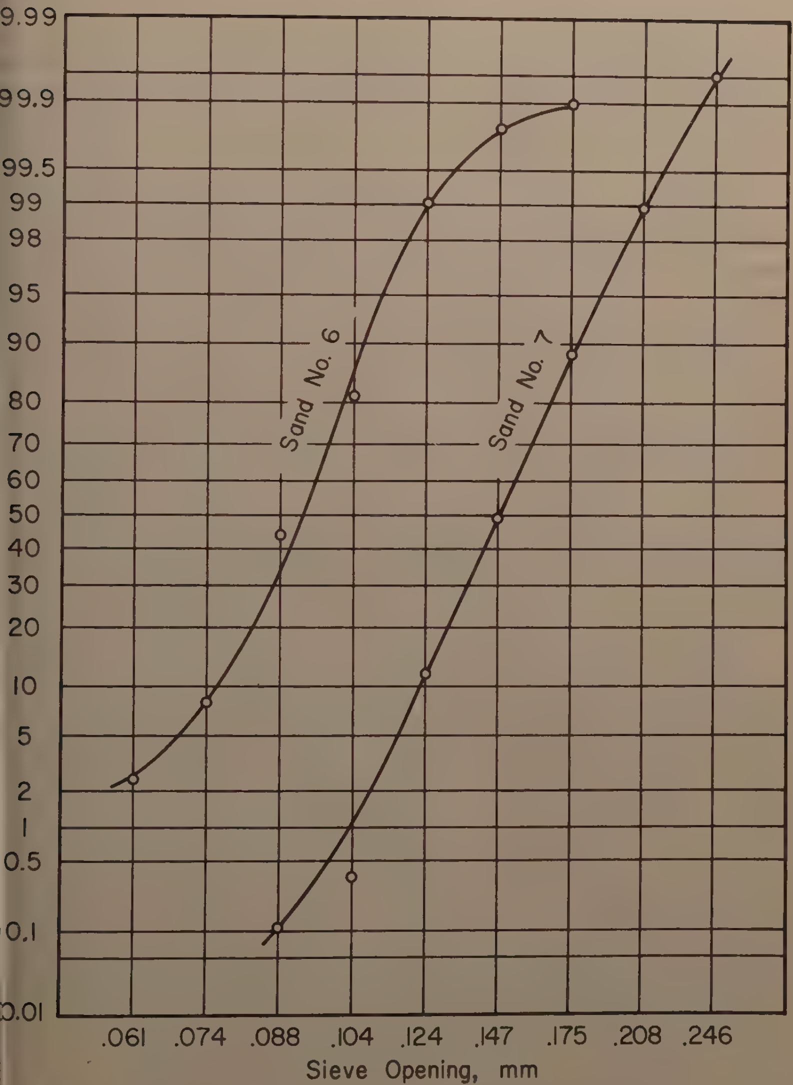


Fig. 3. Cumulative size frequency curves of the sands used in the experiments.

steps followed in the stabilizing process are as follows:

a) Drain the water from the flume and allow the sand to dry for 24 to 48 hours so that the moisture content is about 10 per cent by weight.

b) Spray the sand with a mixture of the following three materials:
1. 68% by volume of a 2% solution of sodium aluminate,
2. 22% by volume of a solution of sodium silicate of specific gravity 1.
3. 10% by volume of pure water.
This mixture was applied to the bed with a paint sprayer until the sand showed signs of saturation. This required an amount of the mixture about equal to 10% by weight of the sand being treated.

c) After drying for about 12 hours, the sand was sprayed with a light application of a calcium chloride solution with specific gravity 1.2.

d) Finally, a thin coat of synthetic varnish manufactured by Krylon, Inc., and sold under the trade name "Krylon Acrylic," was sprayed on the sand to form a waterproof surface.

One of the most difficult problems in the stabilization process was the preservation of the proper grain roughness of the surface of the bed. To disturb the surface characteristics as little as possible, care was taken not apply an excess of chemicals which would flood the surface, and the solutions were applied gently with a paint sprayer aimed parallel to the bed instead of at the bed. Although the results appeared satisfactory visually, there was, unfortunately, no method of testing whether the grain roughness of the finish surface was exactly the same as that of the loose sand itself.

TABLE 3. Relation Between Sedimentation Diameter and Sieve Diameter Based on Vanoni (21) for Nevada White Sand

Tyler Sieve No. retained on	Sieve opening next above mm	Sieve opening retained on mm	Mean sieve diam. mm.	D_{si} Mean sedimentation diam. mm.	w_i Fall velocity at 20°C fps	w_i Fall velocity at 25°C fps
80	0.208	0.175	0.191	0.198	0.078	0.083
100	0.175	0.147	0.161	0.168	0.061	0.066
115	0.147	0.124	0.135	0.153	0.053	0.058
150	0.124	0.104	0.114	0.128	0.038	0.042
170	0.104	0.088	0.096	0.107	0.029	0.032
200	0.088	0.074	0.081	0.096	0.024	0.027
250	0.074	0.061	0.0675	0.086	0.020	0.022

Results

ne of Experiments

he general plan of the experiments may be seen from Table 4, which 's some of the principal quantities measured. The experiments are di- l into sets and individual experiments. The flow depth, discharge, and ı velocity are kept the same in each set of experiments, but the slope is ed. In Sets III and IV the depth, discharge, and velocity are kept constant ghout. Sets I, II, and III were made with a .091 mm sand and a .148 mm was used in Set IV.

ach set contains runs with (i) movable bed, (ii) stabilized bed and clear r, and (iii) stabilized bed with varying amounts of loose sand added to the em where the material added is the same as that in the bed. In Runs 2, 4, d 8, the bed was stabilized and no sediment was being transported by the i.e., the flow consisted only of clear water.

able 4 shows the most important measured and calculated quantities ob- d in the experiments. Where blank spaces occur in the table, the items either not measured or not calculated. Measured values of the exponent the suspended load equation (Eq. 7) are listed in Fig. 8 for Runs 1, 3, 5, 7, which were those with a movable bed.

Configuration

bservations of the bed configuration and the movement of the configuration made visually and by means of systematic photographs during the ex- ments. Fig. 4 shows a side view of the bed for Run 1 and a plan view for 2, where the bed configuration for Run 2 is the same as that for Run 1 pt that it has been stabilized by the method described above. Fig. 5 's a similar set of pictures for Runs 3 and 4. Fig. 6 shows the side view plan view of the bed in Run 5, which has been described as "flat" in e 4. It will be seen that the bed is actually not entirely flat but that e are small ripples near the wall. The pictures of Fig. 4, 5, and 6 illus- the meaning of the terms used to describe the bed condition in the 15th mn of Table 4.

city profiles

elocity profile measurements were made at the centerline of the flume ation 24, i.e., 24 ft downstream from the inlet to the flume. These surements fitted the logarithmic law (Eq. 1) very well, as may be seen g. 7 which shows the profiles for Runs 1, 3, 5, and 6. The quantity m l on the figure is the slope of the lines fitted to the points in fps per e. Eq. 2 can be transposed to give

$$m = \frac{u_2 - u_1}{\log \frac{y_2}{y_1}} = \frac{2.3}{k} \sqrt{\frac{\tau_o}{\rho}} = \frac{2.3}{k} u_*$$

which one gets

$$k = \frac{2.3}{m} u_* \qquad (4)$$

Fig. 4. Bed configuration, Set I, U = 1.23 fps, d = .284 ft.
a) Run No. 1, side view, loose sand, during flow
b) Run No. 2, plan view, stabilized bed, looking upstream, without flow.

Fig. 5. Bed configuration, Set II, U = 2.02 fps, d = .244 ft.
a) Run No. 3, side view, loose sand, during flow
b) Run No. 4, plan view, stabilized bed, looking upstream, without flow.

SUMMARY OF

Set No.	Run No.	Q Discharge cfs	d Depth ft	r Hydr. Radius ft	S Slope	U_* Shear Velocity fps	U Average Velocity fps
							Sand No. 6
I	1	0.306	0.284	0.172	0.0025	0.118	1.23
	1A	0.306	0.284	0.172	0.00245	0.117	1.23
	2	0.306	0.284	0.172	0.0026	0.121	1.23
	2a	0.306	0.284	0.172	0.00255	0.119	1.23
	2b	0.306	0.284	0.172	0.00255	0.119	1.23
	2c	0.306	0.284	0.172	0.0025	0.118	1.23
	2d	0.306	0.284	0.172	0.00245	0.117	1.23
II	3	0.433	0.244	0.157	0.0020	0.101	2.02
	4	0.433	0.244	0.157	0.0025	0.112	2.02
	4a	0.433	0.244	0.157	0.0023	0.108	2.02
	4b	0.433	0.244	0.157	0.0021	0.103	2.02
	4c	0.433	0.244	0.157	0.0020	0.101	2.02
III	5	0.509	0.257	0.162	0.00206	0.104	2.26
	5A	0.509	0.255	0.161	0.00206	0.104	2.28
	6	0.509	0.255	0.161	0.00251	0.114	2.28
	6a	0.509	0.255	0.161	0.00227	0.108	2.28
	6b	0.509	0.254	0.161	0.00227	0.108	2.28
	6c	0.509	0.252	0.160	0.00210	0.104	2.30
							Sand No. 7,
IV	7	0.509	0.255	0.161	0.00258	0.116	2.28
	8	0.509	0.253	0.160	0.00293	0.123	2.29
	8B	0.509	0.253	0.160	0.00259	0.116	2.29
	8a	0.509	0.254	0.161	0.00259	0.116	2.28
	8b	0.509	0.253	0.160	0.00257	0.116	2.29
	8c	0.509	0.253	0.160	0.00257	0.116	2.29
	8d	0.509	0.253	0.160	0.00259	0.116	2.29

Notes: 1. Temperature for all runs: 25.0°C.
2. ** Not measured

Key for Bed Condition:

(1) Dunes

(2) Dunes, largest at walls tapering down to nearly flat in center of flume.

(3) Ripples near walls, flat in center.

(4) Flat except for slight ripples extending about 2 inches out from walls.

RIMENTS

u_{*cl} hear el. at enter ine fps	k_{cl} von Karman const. at Center Line	$\bar{C}$ Sed. Discharge Conc. gm/ℓ	F Froude No.	Bed Condition See Footnotes	Bed Treatment See Footnotes	Amount of Loose Sand in System Kilograms
= 1.16						
0.151	0.369	3.64	0.41	(1)	(a)	90
0.150	0.367	3.38	0.41	(1)	(a)	90
0.155		0.00	0.41	(1)	(b)	0
0.153		0.31	0.41	(1)	(c)	**
0.152		0.47	0.41	(1)	(c)	**
0.151		0.60	0.41	(1)	(c)	**
0.150		0.83	0.41	(1)	(c)	**
0.110	0.265	4.60	0.72	(2)	(a)	85
		0.00	0.72	(2)	(b)	0
		1.71	0.72	(2)	(c)	**
		3.44	0.72	(2)	(c)	**
		3.63	0.72	(2)	(c)	**
0.111	0.223	6.92	0.78	(4)	(a)	55
0.109	0.219	8.08	0.79	(4)	(a)	55
0.136	0.384	0.00	0.79	(4)	(b)	0
0.116	0.255	3.99	0.79	(4)	(c)	3.00
0.116	0.236	5.71	0.80	(4)	(c)	6.00
0.114	0.209	6.82	0.81	(4)	(c)	9.00
= 1.16						
0.135	0.299	3.61	0.79	(4)	(a)	55
0.150	0.364	0.00	0.80	(4)	(b)	0
0.134	0.355	0.00	0.80	(4)	(b)	0
0.135	0.345	0.51	0.80	(4)	(c)	0.50
0.133	0.330	1.41	0.80	(4)	(c)	1.50
0.133	0.319	2.43	0.80	(4)	(c)	3.00
0.134	0.310	3.27	0.80	(4)	(c)	6.00

for Bed Treatment:

(a) Bed of loose sand covering bottom of flume.

(b) Sand bed stabilized in natural configuration with no loose sand.

(c) Sand bed stabilized in natural configurations with loose sand added.

Fig. 6. Bed configuration, Set III, U = 2.28 fps, d = .255 ft.
a) Run No. 5, side view, loose sand, without flow
b) Run No. 5, plan view, looking upstream, loose sand

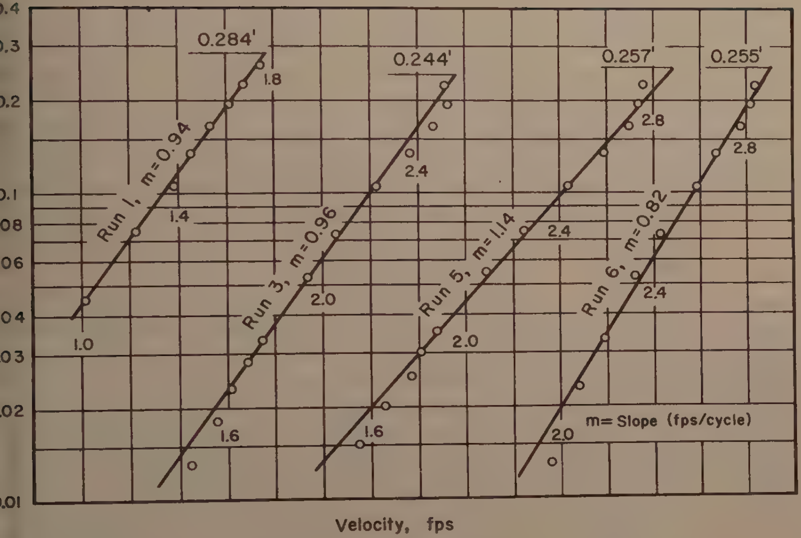


Fig. 7. Measured velocity profiles at the centerline of the flume at Station 24 for Runs 1, 3, 5, 6.

vonKarman constant $k_{c\ell}$ for the centerline profile was determined from

$$k_{c\ell} = \frac{2.3}{m} u_{*c\ell} \tag{5}$$

re the centerline shear velocity $u_{*c\ell}$ is given by

$$u_{*c\ell} = \sqrt{\frac{f}{8}}\ \bar{u}_{c\ell} \tag{6}$$

re $\bar{u}_{c\ell}$ is the mean velocity calculated from the measured profile at the erline and f is the friction factor for the channel obtained from Eq. (3) g the hydraulic radius of the section r instead of r_b. The values of $k_{c\ell}$, ch are listed in Table 4, consistently decrease as the concentration $\overline{C}$ eases.

ment Concentration Profiles

he suspended load equation for distribution of concentration c of parti- with settling velocity w over a vertical is

$$\frac{c}{c_a} = \left(\frac{d-y}{y}\ \frac{a}{d-a}\right)^z \tag{7}$$

re y is the distance up from the bottom, d is the flow depth, c_a is the entration at the level $y = a$, and z the exponent is given by

$$z = \frac{w}{\beta k u_*} \tag{8}$$

where β is the ratio of the diffusion coefficients for sediment and momentum. Fig. 8 shows logarithmic graphs of (d-y)/y versus c for measurements made on the longitudinal centerline of the flume. It is seen that these sediment profiles follow Eq. 7, the well-established equation for distribution of suspended load. The values of the exponent z calculated by Eq. 8 using $k_{c\ell}$ from Table 4, $\beta = 1$, and $u_* = u_{*c\ell}$, (the centerline value defined by Eq. 6) are very close to those appearing in Fig. 8 which are determined from the slope of the lines in the figure. It is of interest to note that the profile for Run 1 agrees with the theory as well as the other runs in spite of the fact that it is the only one of the four for which the bed was covered with dunes.

Friction Factor

From Table 4 it is seen that the bed friction factor f_b is smaller for sediment-laden flow over loose sand beds than for comparable clear flow over fixed beds of the same configuration. The f_b - values for comparable sediment-laden and clear-water flows are summarized in Table 5. This shows clearly the effect of moving sediment in reducing the friction factor, which in these experiments amounted to from 5 to 28 per cent. The results shown in Tables 4 and 5 also indicate that the larger decreases in the friction factor f are associated with larger mean concentrations $\overline{C}$.

Attention is called to the fact that f_b - values for the "identical" runs, Runs 8 and 8B, with clear water and stabilized bed, are not the same. Run 8 was made once and then repeated on the following day, and the results agreed closely. However, when the water had been in the flume for longer than a total time of about 10 hours, some changes occurred, giving the results of Run 8B. To further investigate this problem, Run 8B was repeated three times, once on each of three days, drying the sand bed after each experiment. On the first day the water was run continuously.

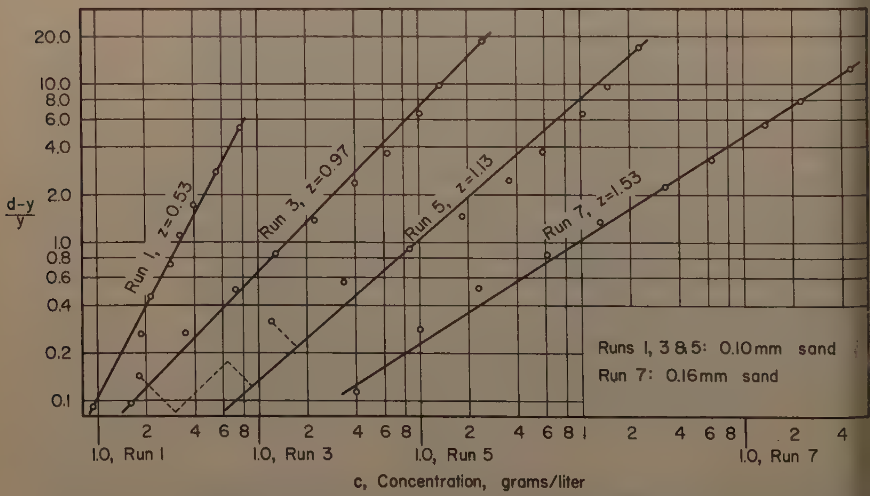


Fig. 8. Measured concentration profiles on centerline of the flume at Station 24 for Runs 1, 3, 5, 7.

On the second day the experiment was completed in as short a time as sible and the pump stopped, leaving the water in the flume. After about ee hours the run was repeated. The flow characteristics and friction fac- of the flow were changed by approximately the same amount as they would e been if the water had been running continuously. The run for the third was a repetition of that for the first, with approximately the same results. m this it was concluded that the water was in some way affecting the var- 1 coat applied to the sand when the water remained in the flume for long iods. Run 8 is considered to give the valid results, with Run 8B being cted by swelling of the varnish between grains.

Discussion of Results

ect of Sediment Load on the Friction Factor

The effect of the sediment load on the friction factor may be seen in Table comparing the results of clear and sediment-laden water for each set of s. Sets I, II, and III were made with the same sediment, so the results directly comparable. Set IV was made with a coarser material. It is r that by adding a sediment load to a flow the friction factor is decreased, e in all four sets of experiments the sediment-laden flows had smaller tion factors than the comparable clear flows. The relative magnitude of reduction in friction factor seems to vary with concentration, the values highest concentration producing the largest change. Referring again to le 5, it is seen that in Set I the friction factor f_b for a flow with a con- ration of 3.64 grams per liter was 5 per cent lower than in the compara- clear flow. In Series II and III the reductions in f_b were 25 per cent and er cent with concentrations of 4.60 and 8.08 grams per liter, respectively. m these figures it appears that rate of change in f_b with concentration de- ses at high values of the concentration.

TABLE 5. Comparison of Friction Factors for Sediment-Laden Flows with Those for Clear Water Flows over Stabilized Sand Beds of the Same Configuration.

Run No.	Depth ft	Bed Cond.	Sed. Disch. Concent. $\bar{C}$ grms/liter	Friction Factors Mean f	Friction Factors For Bed f_b	Decrease Δf_b	Decrease o/o
1	0.284	Dunes	3.64	0.074	0.106	.006	5
2	0.284	Dunes	0	0.077	0.112		
3	0.244	Small dunes	4.60	0.0198	0.0211	.0072	25
4	0.244	"	0	0.0246	0.0283		
5A	0.255	Flat	8.08	0.0165	0.0165	.0064	28
6	0.255	Flat	0	0.0203	0.0229		
7	0.255	Flat	3.61	0.0207	0.0227	.0035	13.5
8	0.253	Flat	0	0.0230	0.0262		

The reduction in f_b may be attributed to alterations of the turbulent structure of the flow by the suspended load. The mechanism of this process is discussed in more detail below.

Effect of Bed Configuration on the Friction Factor

The effect of dunes and ripples on the friction factor agrees with the data obtained by others.(10,11,12) For example, the large dunes in Set I cause f_b to attain the high value of 0.106 in Run 1, compared with the values for all other runs which were less than 0.030 because of predominantly flat beds. From these data one can conclude that in flume experiments changes in friction factor due to dunes can greatly exceed changes due directly to the moving sediment.

In a study of the Mississippi River, Eden(14) observed that at Vicksburg the largest dunes occurred at the highest stage. At Memphis, he observed bed irregularities only on the rising part of the highest stage. The largest of these had heights much as 34 ft. Carey and Keller(15) observed bed configurations of the Mississippi River near Baton Rouge by means of a sonic fathometer. They also found that the largest dunes occurred at the highest stages. There were several systems of dunes of varying wavelengths, the largest of which had lengths of as much as two miles and heights of the order of 30 ft. The shorter dunes were superposed on the larger ones. A study of these profiles showed that at low stage some of the longer waves disappeared but that short waves of the order of 20 to 50 ft in length and 1 to 5 ft in height remained. Fig. 9, reproduced from the work of Carey and Keller, shows profiles at Donaldsonville gage for two stages. For the high stage which occurred on April 10, 1956, it is seen that there are waves of considerable length. For the lower stage which occurred in July, some of the longer waves are no longer present. Despite the increase in dune height with stage, Eden(14) found that the Manning coefficient n decreased as the stage increased. The sediment load also increases with stage, and can account for some reduction in the roughness factor but not necessarily for all of it. It is possible that the long waves actually do not contribute much to the bed roughness and that the shorter, steeper ones, which seem to predominate at the low stages, may actually result in a higher roughness than the larger ones which have been observed at the highest stages.

Data presented by Leopold and Maddock(13,22), an example of which is Fig. 10, for the spring flood of the Colorado River at the Grand Canyon (1941) show that for a given discharge the velocity is higher on the rising than on the falling flood means that the friction factor is lower on the rising flood, assuming insignificant changes in slope and slow rate of change of stage. This follows the same pattern as observed in flume experiments where flows with the higher velocities and sediment concentration tend to have lower friction factors, although it is not clear from the field data whether the damping effect of suspended sediment or change in the bed configuration is the predominant factor in effecting the change.

Leopold and Wolman(23) report that in degrading portions of the Rio Grande, the river channel characteristics follow a pattern similar to that of the Colorado River shown in Fig. 10. In the lower reaches of the river where the bed was aggrading, the velocity and sediment concentration were lower on the rising than on the falling flood, so that the friction factor was not highest on the rising flood. The lower friction factor is still associated with high

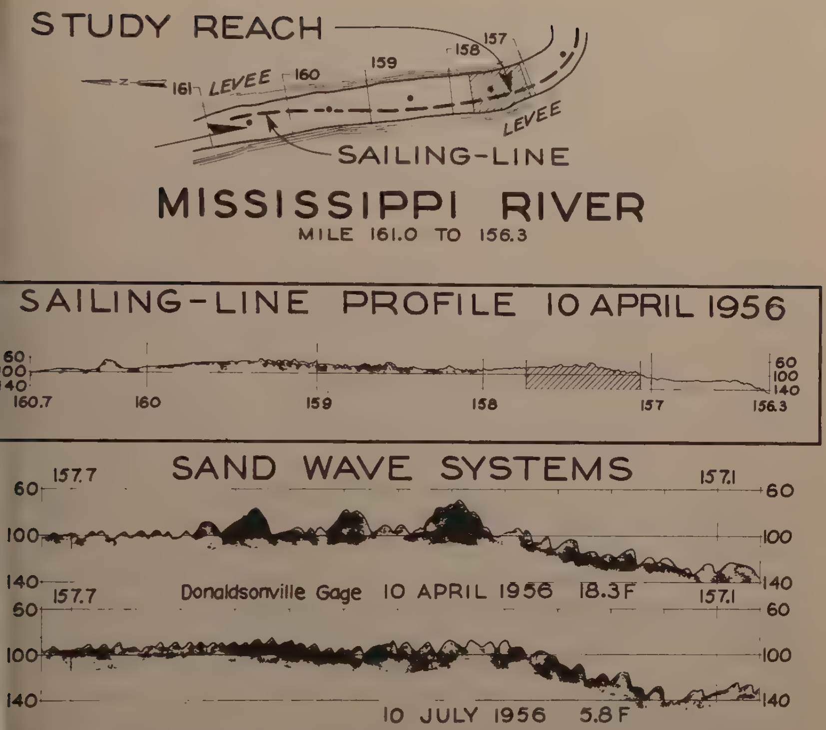


Fig. 9. Bed profiles for lower Mississippi River (Louisiana) from Carey and Keller.(15)

entration, but for some reason the sequence of change has been reversed. e same authors(23) also report that the characteristics of streams in the humid areas, such as in the eastern United States, are much more stable those of the arid western part. In these streams the sediment loads are lower and the beds apparently much more stable, so that the changes in ed are probably much smaller than in streams whose beds are easily ed.

t of Sediment Load on the Velocity Profile

e effect of suspended sediment on the velocity profile of a stream is il- ated by Fig. 2, which shows profiles for a clear flow and sediment-laden with the same depth and slope. It is clear that in addition to having the r velocity at any level, the sediment-laden flow also has a higher veloci- adient. Fig. 2 shows that very near the bed the velocity of the two flows out the same, so that at some distance from the bed the flow with the r velocity gradient will have the higher velocity. By differentiating the ion for the velocity distribution, Eq. 1, with respect to y, one obtains

$$\frac{du}{dy} = \frac{1}{ky}\sqrt{\frac{\tau_o}{\rho}} = \frac{u_*}{ky} \tag{9}$$

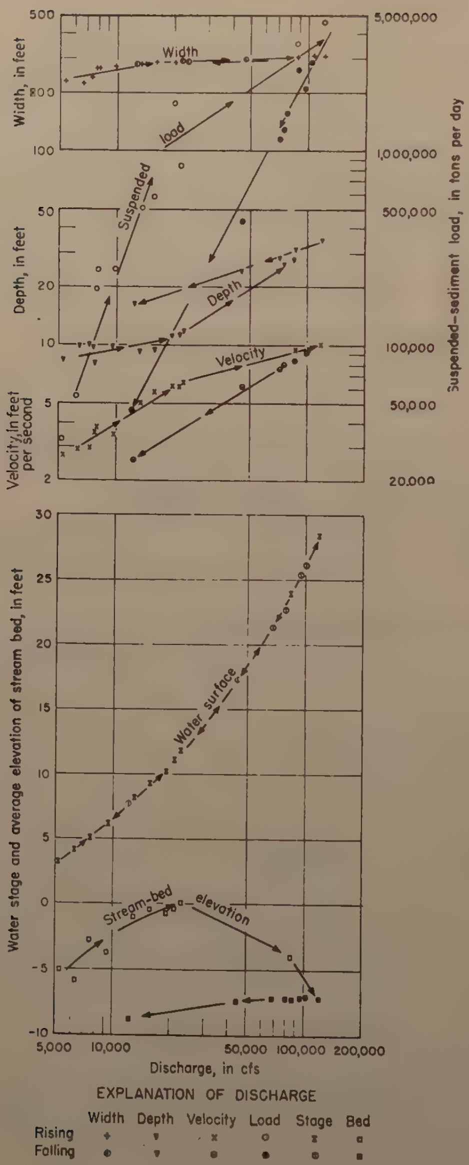


Fig. 10. Changes in width, depth, velocity, stage, and streambed elevation with discharge during flood of December 1940-June 1941, Colorado River at Grand Canyon, Arizona (from Leopold and Maddock).(21)

in the two flows the shear velocity u_* is the same, the gradient for any of y is inversely proportional to k, and since the gradient is seen to ease with load, k must decrease with the load. Thus it can be seen that ffect of sediment on the velocity gradient can be studied by observing k. ne fact that suspended sediment increases the velocity gradient was obed by Gilbert(1) page 229, although the full significance of this was aptly not realized. Flume experiments(7,8) have shown that this is a ematic effect, and observations on the Missouri River by the Corps of neers(24) showed that this effect was also present in natural streams. mechanism by which the sediment increases the gradient, and hence the velocity, has been explained in terms of the damping effect of suspended rial on the turbulence.(7)

keep sediment in suspension, i.e., to prevent it from settling due to itational force, work must be done on the sediment grains. This work come from the vertical components of turbulence fluctuations and must t in damping of the turbulent motion. To investigate this problem, ein and Chien(25) correlated k with the ratio of the power, P_s, to sus- sediment to the power, P_f, to overcome hydraulic resistance to the using data obtained from measurements on the Missouri River and from atory experiments by Ismail.(9) A similar graph was presented(8) for experiments made with 0.1 mm sand. Buckley(6) and Chien,(26) in disng the effect of sediment on the flow, postulated that the main damping t of the sediment on turbulence occurred near the bed where the concenon was highest. This appears reasonable since it is in this zone that the rate is highest and most of the turbulence is produced. With this idea nd, k was correlated against the ratio of the power P_s' to suspend the nent in a thin layer near the bed to the power P_f required to overcome rictional resistance. The power P_s to support sediment in a water n with unit horizontal area and depth d equal to the stream depth is

$$P_s = \frac{\gamma_s - \gamma}{\gamma_s} \bar{c} \, w \, d$$

e γ and γ_s are the specific weights of the water and sediment respec- , $\bar{c}$ is the mean concentration over the depth in weight per unit volume, w is the settling velocity of the sediment. The power P_f to overcome iction of the prism of water is

$$P_f = \gamma \, d \, \bar{u} \, S$$

e S is the slope of the energy grade line and $\bar{u}$ is the mean velocity over epth. The ratio of the two powers is

$$\frac{P_s}{P_f} = \left(1 - \frac{\gamma}{\gamma_s}\right) \frac{\bar{c} \, w}{\gamma \, \bar{u} \, S} \tag{12}$$

e power P_s' to support the sediment suspended between levels $y = y_1$ $= y_2$ in a prism of unit horizontal area is,

$$P_s' = \left(1 - \frac{\gamma}{\gamma_s}\right) \bar{c}_1 \, w \, (y_2 - y_1)$$

where $\overline{c}_1$ is the mean sediment concentration in weight per unit volume between the levels y_1 and y_2. The ratio of P'_s to P_f is

$$\frac{P'_s}{P_f} = \left(1 - \frac{\gamma}{\gamma_s}\right) \frac{\overline{c}_1 \, w}{\gamma \, \overline{u} \, S} \; \frac{y_2 - y_1}{d} \qquad (13)$$

Fig. 11a shows a graph P'_s/P_f plotted against k for flume data obtained with uniform sand of 0.1 mm size, where y_1 and y_2 have been taken as .001 and .01 times the flow depth respectively. Fig. 11b shows a graph of the same data(8) plotted according to the method of Einstein and Chien(25) i.e., k is plotted against P_s/P_f. It is seen that the correlation of the data in Fig. 11a is better than in 11b. This tends to support the ideas of Buckley and Chien and may also explain why the velocity remains logarithmic in a sediment-laden flow, as seen in Fig. 2, even though the turbulence has been modified considerably. If significant damping of the turbulence occurred over the entire depth, one would expect a distortion in the distribution of the turbulence, and hence in the shape of the velocity profile. However, if the principal interference with the turbulence occurs near the bed, the turbulence energy will be reduced but it may still follow the same laws in diffusing through the section, and one might expect the same shape of velocity distribution.

Referring again to Fig. 2, it is seen that the two flows have the same depth and hence, hydraulic radius, and the same slope. The one with the higher mean velocity, i.e., the sediment-laden flow, will have the lower friction factor. The decrease in f, shown in Table 4, for the sediment-laden flows, results from the damping phenomenon discussed above. A comparison of the k-values listed in Table 3 will show that the turbid flows have much the lower values, thus indicating that damping has occurred.

General Discussion

The present experiments have shown that a sediment-laden stream acts in two ways to change its roughness coefficient: (1) a smoothing-out of the bed configurations tends to reduce the coefficient as the velocity increases; damping of the turbulence by suspended sediment tends to increase the velocity gradient so that the difference between the velocity near the bed and near the surface increases along with the mean velocity. In the present laboratory experiments with 0.091 mm sand, the change in the bed friction factor f_b resulting from change in bed configuration alone was as much as five-fold in going from a dune-covered bed to a flat bed. On the other hand, the maximum reduction in f_b due to the sediment load was only 28 per cent (Run 5A). Since the Manning roughness coefficient n is approximately proportional to $\sqrt{f_b}$, the corresponding changes in n are about twofold and 13 per cent, respectively. It is significant that the run which showed the maximum reduction in friction factor, i.e., Run 5A, also had the lowest value of k as determined from the centerline velocity profile. From these few data, it appears that the change in bed configuration has a much larger influence on the roughness coefficient than the damping effect. From this one would judge that the large reductions in roughness coefficients reported by Buckley for the Nile, Eden for the Mississippi, and Lindley for the Indus River, could not be the result only of damping, but must have been also affected by significant changes in dunes on the bed. Judging from the flume experiments one would expect the

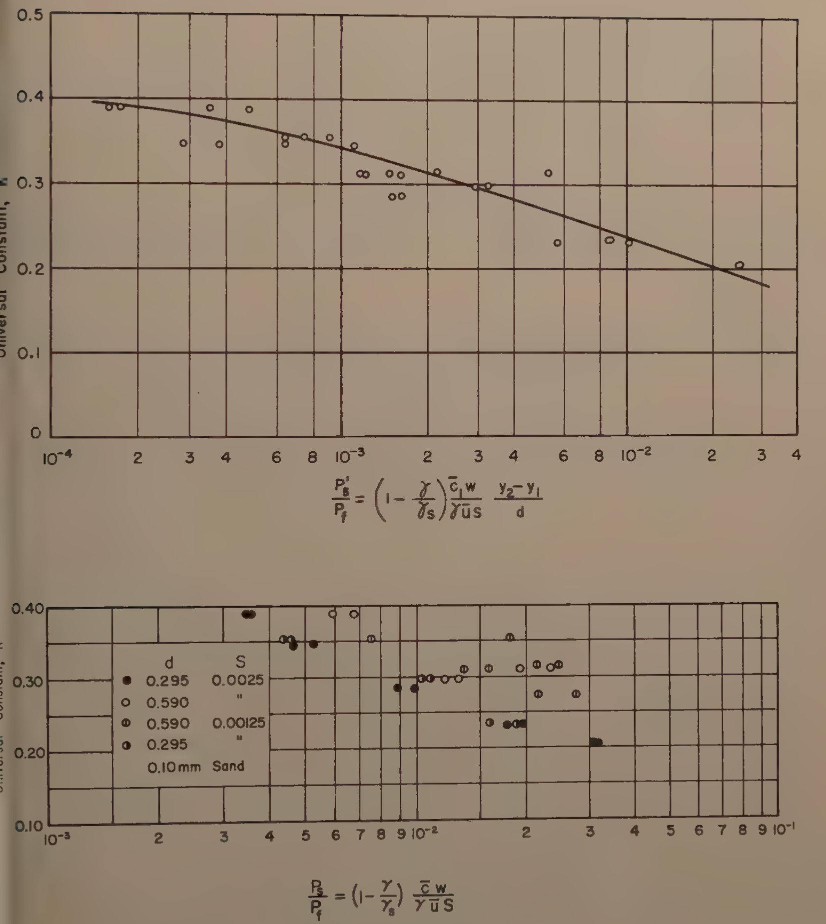


Fig. 11. Reduction of the von Karman constant in sediment-laden flow.

to diminish as the velocity increased, while, at the same time the load ased. Since both of these effects tend to reduce the roughness coeffi- the field observations are in agreement with flume studies and the ss can be explained at least qualitatively by the combination of the two s.

e explanation of the observations that for the same discharge the rough- coefficient of a stream on a rising flood is smaller than on a falling presents some difficulties. Buckley and Lindley noted that for a given ne concentration was higher during the rising flood than during the fall- od, and explained the reduction in roughness by the effect of the higher

load. Judging from the results of the present experiments as shown by Table 5, the changes in roughness factor observed in the field are too large to be accounted for by damping effect alone, and some readjustment of the be must be involved in order to achieve the changes observed. During the risin stage the stream bed probably becomes smoother because the transportation of a high sediment load requires a lower depth and higher velocity compared with the falling-stage flow for which the transport rate is usually less; however, since the damping effect also reduces the roughness, it is not possible to determine the relative magnitude of the two effects from the field data alone. Bottom configuration data of the kind obtained by Carey and Keller, along with stage, slope, discharge and load data, would do much to clarify this problem.

The field data of Buckley, Lindley, and Leopold and Maddock on the variation of stream characteristics, report only total sediment concentration and do not give size distribution of the load or division between wash load and be material load. As seen from Fig. 11, the effect of concentration on k, and hence upon friction factor, depends on the settling velocity w as well as the concentration, so that for the same $\overline{C}$, coarse material with high settling velocity has a greater effect on k and on the reduction in friction factor. By the same token, fine materials such as silt and clay have low settling velocities, and hence small damping effects even for large concentrations. The loads in streams such as the Indus, Colorado and Rio Grande, which show a correlation between mean concentration and velocity (or friction factor)[6,13 are predominantly fine sediments which should have small damping effects, yet changes in load are accompanied by substantial changes in friction factor for the same discharge. According to laboratory results it is unlikely that al of this is caused by damping effects and changes in bed configuration are probably involved.

The discussion thus far has dealt with those observations that show a reduction in friction coefficient as the load increases; however, it will be recalled that Kantlack[6] reported that Swiss engineers found that the velocity was reduced by an increase in the load, and that Latham[3] reported a similar behavior in a storm sewer. The latter might be explained by postulating that dunes were formed when sediment was being transported, thus causing a high friction factor, and that during clear flows sediment was cleaned out of the sewer, leaving the water in contact with the smooth walls, which resulted in low friction at low flow. The explanation of the behavior of the Swiss rivers may be associated with a phenomenon observed by Brooks[27] in a flume at low transportation rates. He observed that there was a tendency at low velocities (i.e., low transportation rates) for the friction factor to increase with the velocity to a maximum, and then to decrease as the velocity increased further. The experiments which showed these results were made in a flume 33.5 in. wide. Two similar sets of experiments made by Brooks in the 10.5-in. flume used in the present experiments did not yield a maximum for f_b but showed f_b decreasing as the velocity increased for all values of the velocity. Therefore, the explanation offered above, based on this one set of experiments, must be considered as speculative.

CONCLUSIONS

Four sets of runs were made in a flume 10.5 inches wide in an effort to separate the direct effect of suspended sediment on the friction factor from

f the bed configuration. First, a natural run was established over a bed of fine sand, and then, after draining the water, the bed was chemi- solidified in its naturally formed configuration. Loose sand was re- d from the system then and a flow of clear water was run over the ized bed, with the usual measurements being taken. Following that, increments of sand were added, noting the change in the friction factor resulted. From these experiments the principal conclusions are as ʼs:

Suspended sediment reduces the friction factor of a flow. In a limited series of laboratory experiments, the amount of this reduction varied from 5 to 28 per cent.

The changes in the friction factor due to variation in bed configuration are much larger than those due to the suspended load itself. A friction factor may change by a factor of 3 to 6 in the laboratory flume between runs of differing velocity at constant depths.

Evidence indicates that the suspended load reduces the friction factor by damping the turbulence or interfering with its production near the bed where the sediment concentration is the highest and the rate of turbulence production is the greatest.

As observed by previous investigators, the von Karman constant k is also substantially reduced by the suspended sediment.

In the field it is impossible to distinguish between the two above- mentioned factors affecting alluvial channel roughness. However, it has been established that the reduction due to the suspended sediment is of importance only for streams carrying a very high suspended load over a flat bed, and is of minor importance when there are dunes on the bed.

ACKNOWLEDGMENT

e research work upon which this paper is based was sponsored by the uri River Division, Corps of Engineers, U. S. Army.

REFERENCES

bert, G. K., "The Colorado Plateau Province as a Field for Geological dy," Amer. Jl. of Science, July-Aug. 1876.

ɔker, Elton Huntington, "The Suspension of Solids in Flowing Water," ıns. Amer. Soc. of Civil Engineers, Vol. 36, p. 239, Dec. 1896.

lham, Baldwin, Minutes of Proceedings Institute of Civil Engineers, . 71 (1886), p. 46. See also Ref. 2, p. 288.

Math, R. E., "Silt Movement by the Mississippi," Van Nostrand's En- eering Magazine, 1883, p. 36. See also footnote Ref. 2, p. 289.

bert, Grove Karl, "The Transportation of Debris by Running Water," S. Geological Survey, Prof. Paper 86, 1914.

6. Buckley, A. B., "The Influence of Silt on the Velocity of Water Flowing i Open Channels," Minutes of Proc. of Institute of Civil Engineers, Vol. 2 p. 183, 1922-23, Pt. II.

7. Vanoni, Vito A., "Transportation of Suspended Sediment by Water," Tra A.S.C.E., Vol. 111, p. 67-133, 1946.

8. Vanoni, Vito A., "Some Effects of Suspended Sediment on Flow Characteristics," Proc. 5th Hydr. Conf., State Univ. of Iowa, Studies in Engine ing, Bul. 34, 1953.

9. Ismail, Hassan M., "Turbulent Transfer Mechanism and Suspended Sediment in Closed Channels," Trans. A.S.C.E., Vol. 117, 1952, p. 409.

10. Brooks, Norman H., "Mechanics of Streams with Movable Beds of Fine Sands," Proc. A.S.C.E., Vol. 81, Sep. No. 668, April 1955.

11. Ali, Said M., and Albertson, Maurice L., "Some Aspects of Roughness i Alluvial Channels," Dept. of Civil Eng., Colorado A. and M. College, Fo Collins, Colorado, Aug. 1953, revised Aug. 1956.

12. Barton, James R., and Lin, Pin-Nam, "A Study of Sediment Transport i Alluvial Channels," Rep. No. 55 J.R.B.2, Civil Eng. Dept., Colorado A. and M. College, Ft. Collins, Colorado, March 1955.

13. Leopold, Luna B., and Maddock, Thomas Jr., "Relation of Suspended Sediment Concentration to Channel Scour and Fill," Proc. of 5th Hydr. Conf., State Univ. of Iowa, Studies in Eng., Bul. 34, 1953.

14. Eden, Edwin W., "A Study of Bed Movement and Hydraulic Roughness Changes in the Lower Mississippi River," M.S. Thesis, Dept. of Mecha and Hydraulics, State University of Iowa, June 1938.

15. Carey, Walter C., and Keller, M. Dean, "Systematic Changes in the Be of Alluvial Rivers," Jl. of Hydraulics Div., A.S.C.E., Vol. 83, No. HY4, Aug. 1957.

16. Harrison, A. S., "Study of Effects of Channel Stabilization and Navigati Project on Missouri River Levels: - Memo. No. 18, Sediment Characteristics of the Missouri River, Sioux City to the Mouth," Omaha Distr Corps of Engineers, U.S. Army, unpublished report, Jan. 1954.

17. Nomicos, George N., "Effects of Sediment Load on the Velocity Field a Friction Factor of Turbulent Flow in an Open Channel," Ph. D. Thesis, Calif. Inst. of Technology, Pasadena, Calif., 1956.

18. Vanoni, Vito A., and Brooks, Norman H., "Laboratory Studies of the Roughness and Suspended Load of Alluvial Streams," Sedimentation Laboratory, California Institute of Technology Report No. E-68, Decem ber 1957.

19. Johnson, J. W., "The Importance of Side-Wall Friction in Bed-Load Investigations," Civil Eng., Vol. 12, No. 6, June 1942, pp. 329-331.

20. Brooks, Norman H., "Laboratory Studies of the Mechanics of Streams Flowing over a Movable Bed of Fine Sand," Ph. D. Thesis, Calif. Inst. Tech., 1954.

anoni, Vito A., "Experiments on the Transportation of Suspended Sedi-
ıent by Water," Ph. D. Thesis, Calif. Inst. of Tech., 1940.

eopold, Luna B., and Maddock, Thomas, Jr., "The Hydraulic Geometry
f Stream Channels and Some Physiographic Implications," U.S. Geologi-
al Survey, Professional Paper 252, 1953.

eopold, Luna B., and Wolman, M. Gordon, "Floods in Relation to the
iver Channel," Publ. No. 42, International Assn. of Hydrology, Dijoh,
'rance, 1956.

. S. Engineer Office, Omaha, Nebraska, "Sediment Characteristics Study
f Missouri River at Omaha, Nebraska," unpublished report, 1951.

instein, H. A., and Chien, Ning, "Second Approximation of the Suspended
oad Theory," Univ. of Calif., Series 47, Issue No. 2, Berkeley, Calif.,
an. 31, 1952.

hien, Ning, "The Present Status of Research on Sediment Transport,"
'rans. A.S.C.E. Vol. 121, 1956, p. 833.

rooks, Norman H., Closing Discussion on Reference 9, Journal of
Iydraulics Div., A.S.C.E., Vol. 83, No. Hy2, Paper 1230, April 1957.

Journal of the

HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

GRAVEL BLANKET REQUIRED TO PREVENT WAVE EROSION

Enos J. Carlson,[1] M. ASCE

SYNOPSIS

rface wave erosion tests were performed in a hydraulic wave flume on rials shipped from Yakima Project, Washington. The tests were made to mine the cover blanket needed to prevent leaching of fine base material. lar talus and rounded gravel materials were shipped to the laboratory the field, together with fine base material to make the tests.

ne most stable cover blanket was formed by screening the talus material 3/4-inch screen. The minus 3/4-inch material being placed over the material with the plus 3/4-inch material on the surface. Motion pic- were taken for a record of the 14 tests.

INTRODUCTION

me factors which must be considered in the design of unlined and earth canals are those which may cause erosion and, consequently, develop n extensive maintenance problem. They include: excessive tractive s which are caused by high velocities and, consequently, result in ero- over a large part of the wetted perimeter; and those forces caused by ce waves which produce erosion on the banks at the water surface. The t and rate of erosion produced by these forces which exist in many un- and earth lined canals designed and constructed by the Bureau of Recla- on, depend upon the type and gradation of the soil material through which anals are constructed.

ne Kinnewick Main Canal, feature of the Bureau of Reclamation's Yakima ct in the state of Washington, was constructed in an area where the ma- is fine and silty and has little cohesion. It is described as a highly

Discussion open until October 1, 1959. To extend the closing date one month, a itten request must be filed with the Executive Secretary, ASCE. Paper 2021 is rt of the copyrighted Journal of the Hydraulics Division, Proceedings of the nerican Society of Civil Engineers, Vol. 85, No. HY 5, May, 1959.

ad, Sediment Investigations Unit, Hydr. Lab., Bureau of Reclamation, nver, Colo.

compressible micaceous, nonplastic silt which had its origin in old lake beds When dry, it is easily crumbled into a powdery dust. Over 70 percent of the silty material passes the No. 200 United States standard screen. This type of material, combined with the prevalent wind in the area, provided the conditions for possible erosion on the canal banks due to waves created by the wi The prevailing direction of the wind is generally west to northwest and is in the same general direction as a large part of the canal. The many curves which were necessary on the canal, constructed in hilly topography, are especially vulnerable to erosion caused by surface waves. It was decided that some type of protective cover over the fine material for the Kennewick Main Canal was essential, particularly on the curves. A hydraulic model study wa made to help determine the adequacy of the available materials to be used as a cover blanket, and determine the method of placement that would give the best protection for the least amount of material and cost.

The Kennewick Main Canal is located in Benton County, Washington, near the Yakima River, between the towns of Prosser and Kennewick (Figure 1). The canal is approximately 42.3 miles in length and is designed to carry a discharge of from 500 to 275 cfs. The unlined and earth lined sections have a 14-foot bottom width and side slopes of 2:1. The depth varies from 7.61 feet to 5.66 feet, corresponding to the discharges cited above. Construction of the canal provides irrigation for approximately 14,500 irrigable acres of new land and additional irrigation for approximately 4,600 acres of land in the area that had been under irrigation. The design capacity of the canal wi permit eventual irrigation of approximately 6,100 acres of additional land.

Hydraulic Model

As shown in Figure 2, a 2:1 slope of plywood was built in an existing flum which was 2 feet wide, 6 feet deep, and approximately 24 feet in length. To make a maximum number of tests with the material which had been shipped from the field to the Bureau's Hydraulic Laboratory in Denver, a plywood divider was placed on the slope, dividing it into two sections. The 1-foot-wide area on the side of the plywood divider next to the glass observation window was used for placing the test section. On the other side of the divid a gravel wave absorber was placed to reduce reflections from this area.

A wave machine, Figure 3, consisted of a bulkhead hinged at the bottom o the flume and moved back and forth about the hinge by an arm connected to a large pulley wheel. The pulley wheel had holes drilled at various radii from the center, thus giving a stroke to the wave-making bulkhead which could be varied from 2-1/8 to 11 inches. A 3/4-hp motor connected to a set of variable speed pulleys completed the wave-making apparatus. With this setup, it was possible to change the frequency of waves from 6 to 315 waves per minute.

In these model tests, the waves were made to move at right angles to the canal slope, and resulting beaching action was more severe than the same height of wave would have been in a prototype canal where the waves would move longitudinally along the side slopes. However, it was possible to compare the various test setups with this condition and thereby select the best test arrangement. With the various setups on the wave-producing machine, it was possible to obtain wave heights up to 0.75 foot from trough to crest. Prior to operation of the wave machine, the model was filled slowly to the

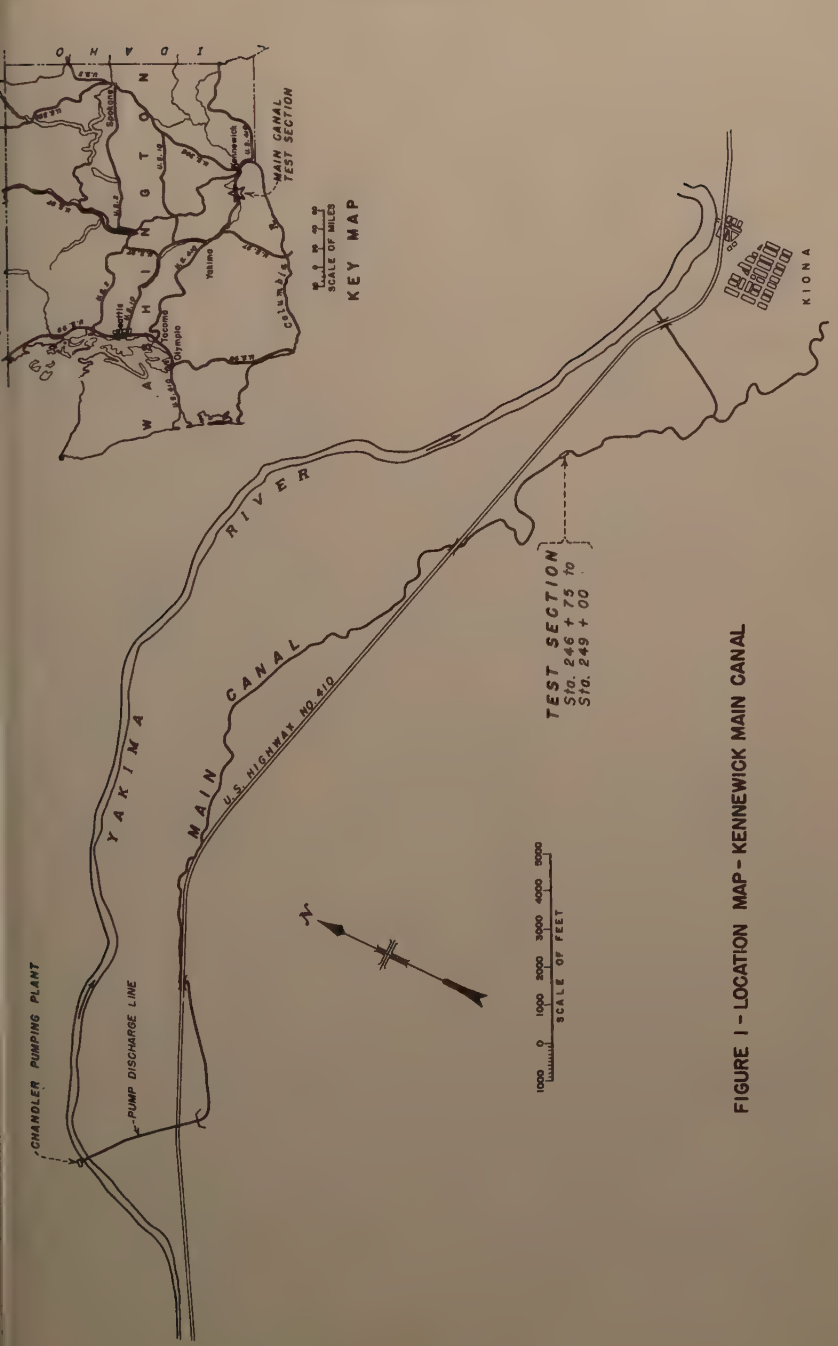
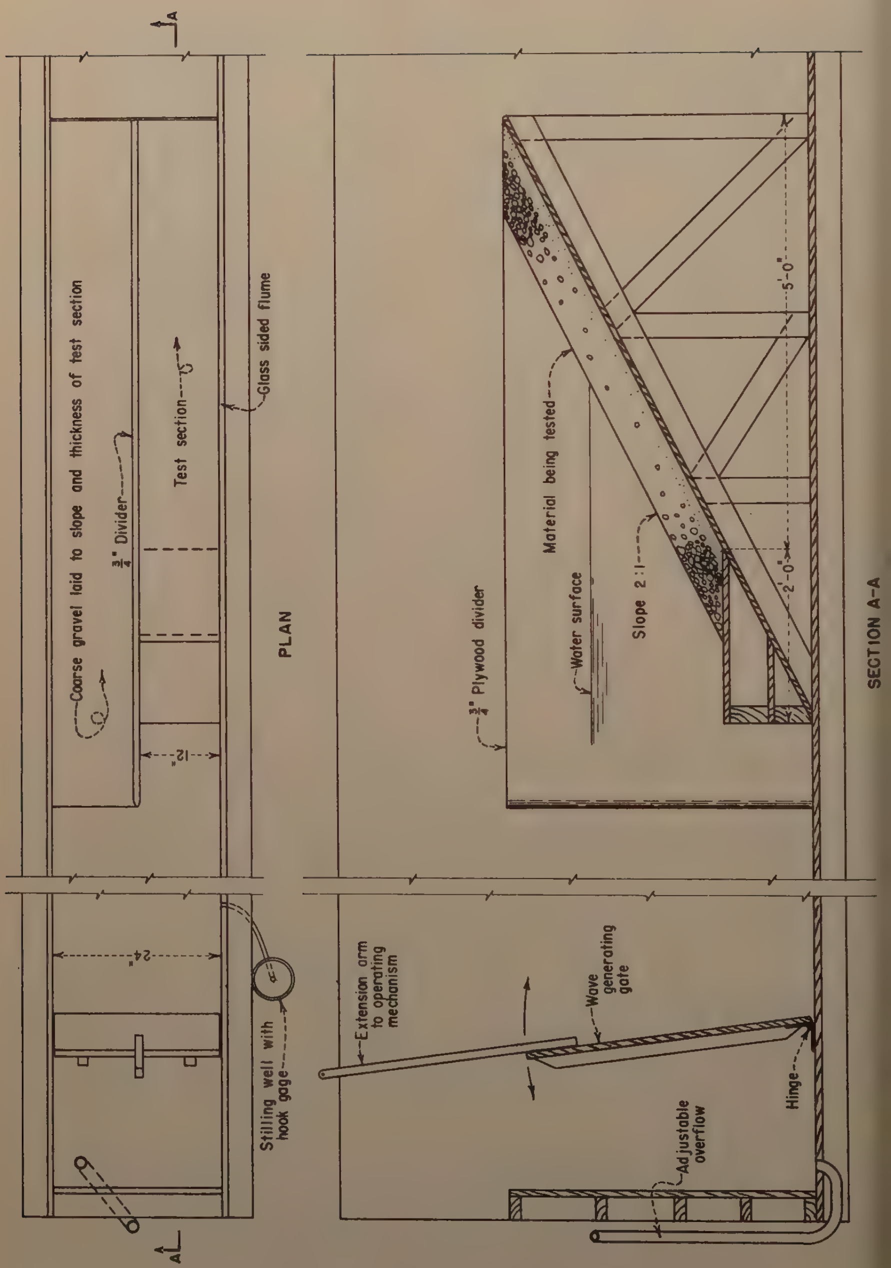


FIGURE 1 - LOCATION MAP - KENNEWICK MAIN CANAL


Coarse gravel laid to slope and thickness of test section
3/4" Divider
Test section
Glass sided flume
12"
24"
Stilling well with hook gage
PLAN
A
A
3/4" Plywood divider
Material being tested
Water surface
Slope 2:1
5'-0"
2'-0"
Extension arm to operating mechanism
Wave generating gate
Hinge
Adjustable overflow
SECTION A-A

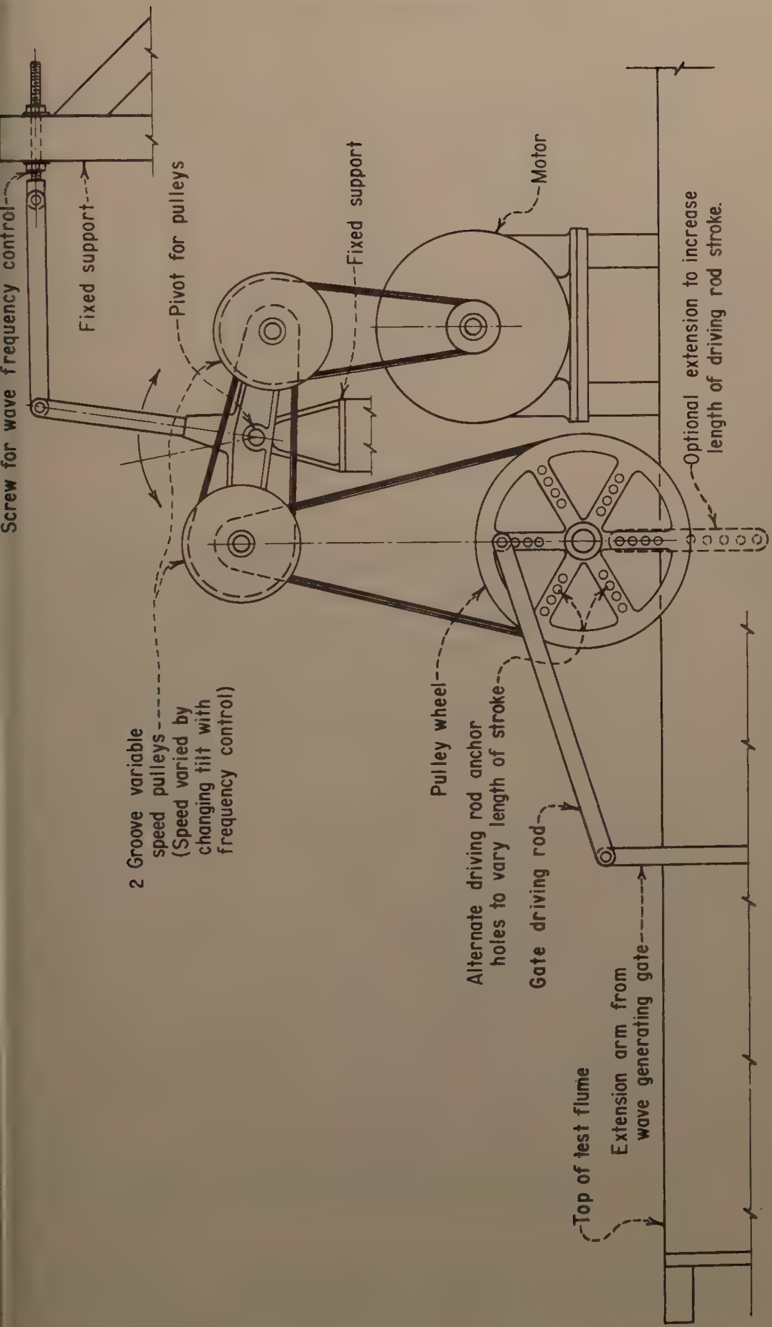


FIGURE 3.- WAVE MACHINE DRIVE AND CONTROL MECHANISM

desired depth which was maintained constant during a test. Water surface elevations were determined by using a hook gage in a well outside the wave area of the model.

For each test, both still and motion pictures were taken as an aid in recording the data from the model. The test procedures were designed to determine which cover material available on the project would give the best protection for the fine silty base material and to determine which would be the most efficient way to place the material to give the best resistance to erosion and least leaching of the base material and still at the minimum cost. In the early tests, a capacitance-type water surface indicator, together with a recorder, was used to determine the type of wave which should be produced as a standard for all tests. After recording the various waves that were produced by adjusting the wave machine to various strokes and frequencies, standard wave patterns were set, and the recording apparatus was no longer used.

Test Slope

Filter Criteria

On the project there were two materials available which could be used as a cover blanket for the Kennewick Main Canal. Both were equally available and in adequate supply for the canal job. Both of these materials fell largely outside the Bureau of Reclamation filter criteria for the base material,[1,2] as shown on Figures 4 and 5. The placing of a cover blanket over finer base material on a canal bank is similar to the filter problem for the foundation of engineering structures to allow drainage and prevent leaching of the base material. The following criteria are used by the Bureau of Reclamation for filters in foundations work.

a. The filter material should be more pervious than the base material to assure there will be no hydraulic pressure built up to disrupt the filter and adjacent structures.

b. The voids of the in-place filter material must be small enough to prevent base material particles from penetrating the filter which causes clogging and failure of the protective filter system.

c. The layer of the protective filter must be sufficiently thick to provide a good distribution of all particle sizes throughout the filter and also to provide adequate insulation for the base material where frost action is involved.

Results of a large series of tests resulted in the following requirements for a graded filter. A graded filter material is defined in this paper as a material having a comparatively broad range of particle sizes. Graded material may have concave, convex, S-shaped or straight line gradation curves, and may be defined as "poorly" and "well-graded" material, depending on their gradation curve shape. Uniform grain-size filters have a similar set of requirements.

For graded filters, it is necessary that the ratio of the 50 percent grain size of the filter material to the 50 percent grain size of the base material be between the limits of 12 and 58

$$R' = \frac{\text{50 percent size F.M.}}{\text{50 percent size B.M.}} = 12 \text{ to } 58$$

ratio of the 15 percent grain size of filter material to the 15 percent n size of the base material should be between the limits of 12 to 40

$$R'' = \frac{\text{15 percent size F.M.}}{\text{15 percent size B.M.}} = 12 \text{ to } 40$$

n addition, the following requirements for a graded filter should be met:

. The filter material should pass the 3-inch screen for minimizing parti- segregation and bridging during placement. Also, filters must not have e than 5 percent minus No. 200 particles to prevent excessive movement nes in the filter and into drainage pipes causing clogging.

. The gradation curves of the filter and the base material should be ap- ximately parallel in the range of finer sizes, because the stability and per function of protective filters depend upon skewness of the gradation ve of the filter toward the fines, giving a support to the fines in the base erial.

. The filter material adjacent to the drainage pipe should have sufficient rse sizes to prevent movement of filter material into the drainage pipe. maximum size of the perforations or joint openings of the drainage pipe selected as 1/2 of the 85 percent grain size of the filter material.

. In designing filters for base materials containing particles larger than 4 size, the base material should be analyzed on the basis of the gradation aterial smaller than No. 4 size.

erials Tested

he gravel, Figure 4, was rounded river-deposited material. Twenty ent of this material was smaller than the No. 30 United States standard e, and 80 percent was smaller than the 1-1/2-inch size. Between the 30 and the No. 4 sieves, there was only 4 percent of the material. Be- e of the dearth of material in this range, the gradation fell largely out- the limits of the graded filter criteria for the base material. The other erial available for a blanket cover was an angular talus found at the base cal mountain slopes. This material was obtained from two locations. combined material gives a gradation of 10 percent smaller than the No. nited States standard sieve, and 80 percent smaller than the 1-1/2-inch ing. The talus material was better graded than the rounded gravel ma- l; however, it also fell outside the limits for a graded filter criteria for ase material, as shown on Figure 5.
gradation of the base material, Figure 6, showed 6 percent finer than 5 mm, 27 percent finer than 0.037 mm, 74 percent finer than 0.074 mm, a small amount of material as large as 3 to 5 inches. Several tests to rmine the liquid limit and plastic limit showed the base material to be lastic.

Hydraulic Wave Tests

1—Gravel Moistened and Well Mixed

he first test was conducted on the gravel material, Figure 7. Six inches e material moistened and well mixed was placed uncompacted on the 2:1

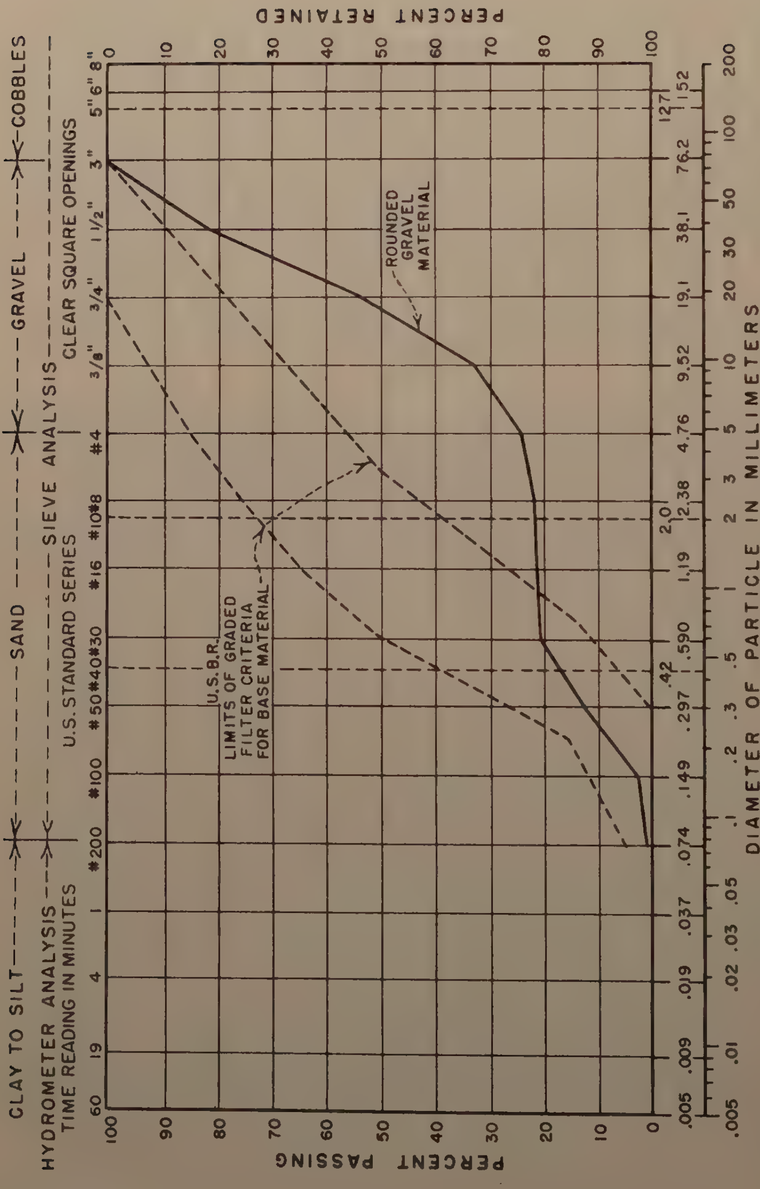


FIGURE 4.— AVAILABLE GRAVEL BLANKET MATERIAL

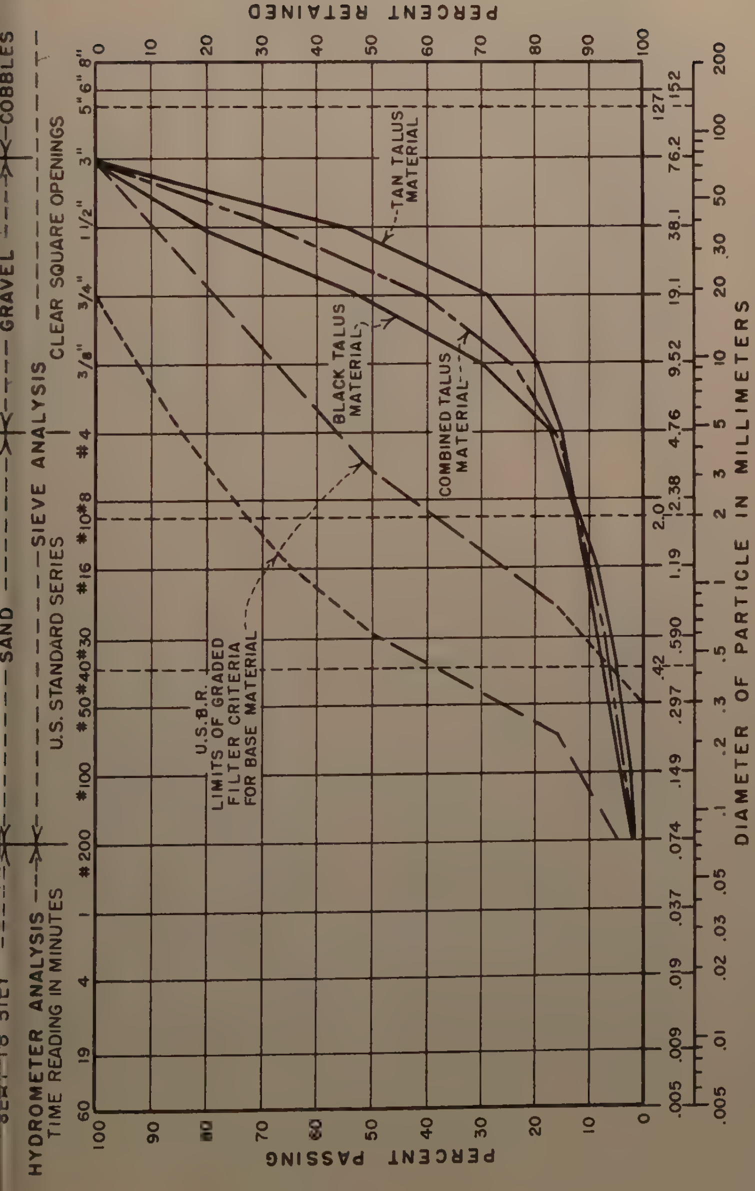


FIGURE 5.—AVAILABLE TALUS BLANKET MATERIALS

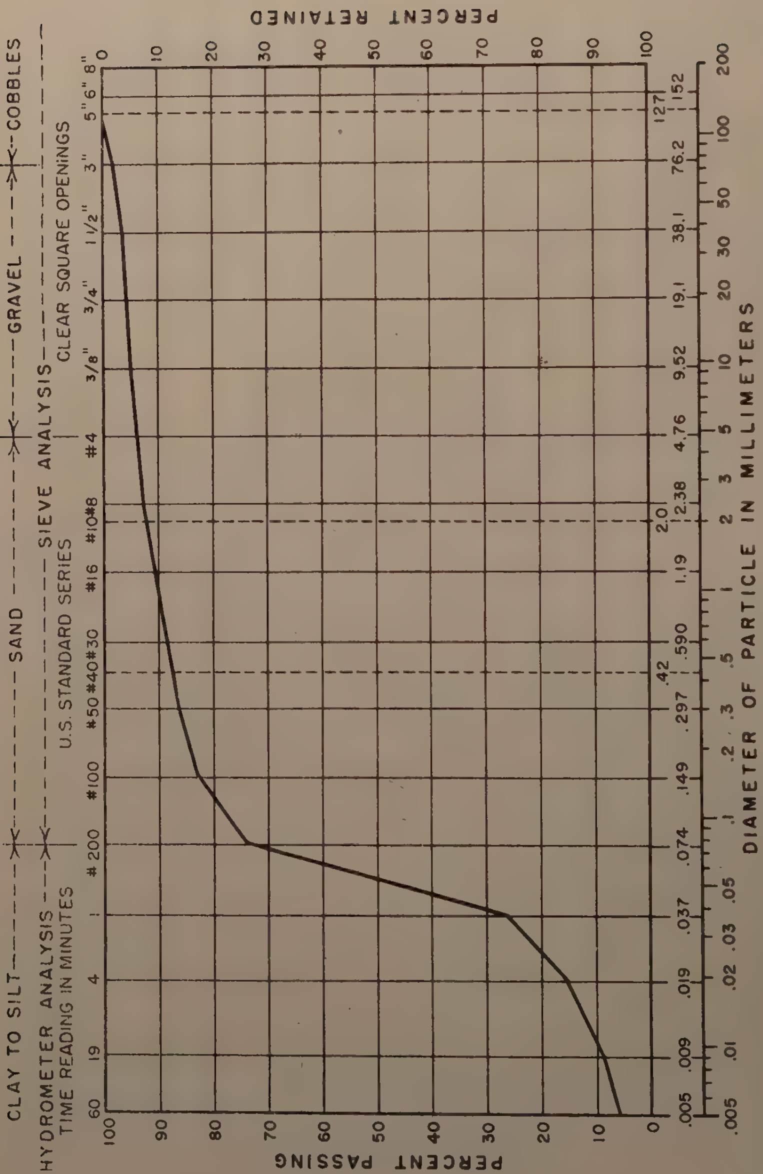


FIGURE 6.—BASE MATERIAL THROUGH WHICH THE CANAL PASSES

Figure 7

(a) Model prepared for Test 1

(b) Slope failure after 45 minutes of operation at 32 waves per minute - wave height 0.32 ft.

Yakima Project - Washington
KENNEWICK MAIN CANAL
TEST 1 (GRAVEL MOISTENED AND WELL MIXED)
Model study

slope. The moist sand particles adhered to the gravel and were well distributed throughout the 6-inch layer. The depth of water was raised to 2.7 feet, and the wave machine was operated at 32 waves per minute. This produced a wave having a height from trough to crest of 0.32 foot. Immediately after the wave machine was started, the gravel and sand material began to slip and roll down the slope. The sand which had adhered to the gravel particles began washing from between the particles, reducing the friction throughout the layer, and the force of the water flowing out of the blanket when each wave receded from the slope was enough to start the particles to slide and roll. Approximately 20 minutes after the beginning of the test, the board slop was exposed. The photograph in Figure 7b shows Test 1 after the test had been in operation for 45 minutes. Complete failure near the water surface had occurred. It seems possible that if the water surface were raised and lowered a few times, allowing the sand particles to wash from between the gravel particles, that a greater coefficient of friction could be obtained throughout the gravel. This cover layer then might have been more stable. The fine sand particles acted as ball bearings, allowing the larger gravel particles to roll and slide, causing failure of the cover blanket.

Test 2—Gravel Separated on 3/4-inch Screen

The gravelly material used in Test 1 was dried and separated on a 3/4-inch screen. The resulting analysis curve and Bureau of Reclamation filter criteria are shown in Figure 8. The analysis curve for the fine base material, minus 3/4-inch material, and plus 3/4-inch material are shown. Filter criteria limits for the base material and the minus 3/4-inch material are shown on the same graph. The graph shows that the minus 3/4-inch material falls roughly within the limits of the graded filter criteria for the base material, except that there is very little material between the No. 4 and No. 30 sieve sizes. The 3/4-inch material falls within the limits of the uniform filter criteria for the minus 3/4-inch material.

A 4-inch layer of the material smaller than 3/4-inch size was placed on the 2:1 slope and covered by a 4-inch layer of material larger than 3/4-inch in size. The water was raised to a depth of 2.7 feet, and a wave frequency was set at 32 waves per minute. The resulting wave height was 0.32 foot. Th model, ready for operation, may be seen in Figure 9a. After 1 hour's operation at these conditions, there was practically no observable damage on the cover blanket. The wave frequency was then increased to 42 per minute, wit the water depth maintained the same. This increase in frequency caused a wave height to increase from 0.32 to 0.43 foot.

After operating the model for 23 hours at 42 waves per minute, there was only a small amount of beaching. It was believed the resulting damage, show on Figure 9b, was not sufficient to cause failure in a canal installation. Ther was some movement of sand in the gravel below the 3/4-inch layer. The san appeared to move down the slope through the gravel in the area where the tw layers were in contact. As may be seen by studying the photographs (Figure 9), the sand moved in the top 1-1/2 inches of the 4-inch layer of minus 3/4-inch material. Some of the sand in the material smaller than 3/4-inch in siz concentrated at the bottom of the layer, thus, probably improving the action c material and preventing beaching.

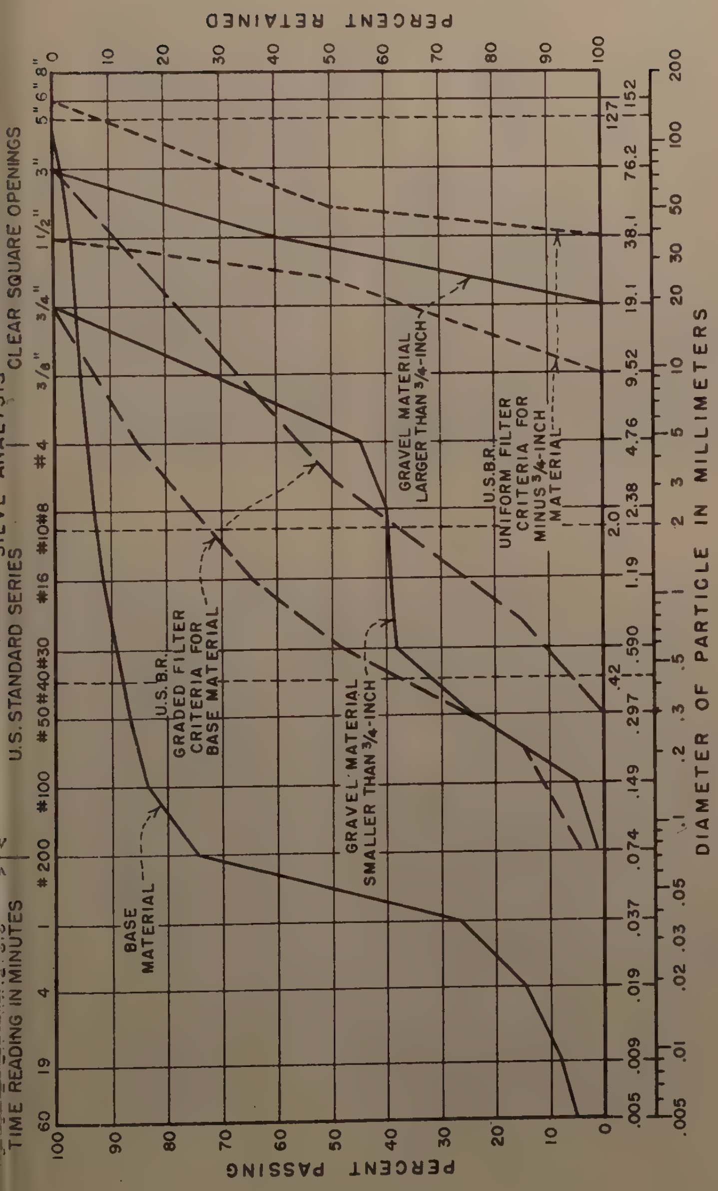
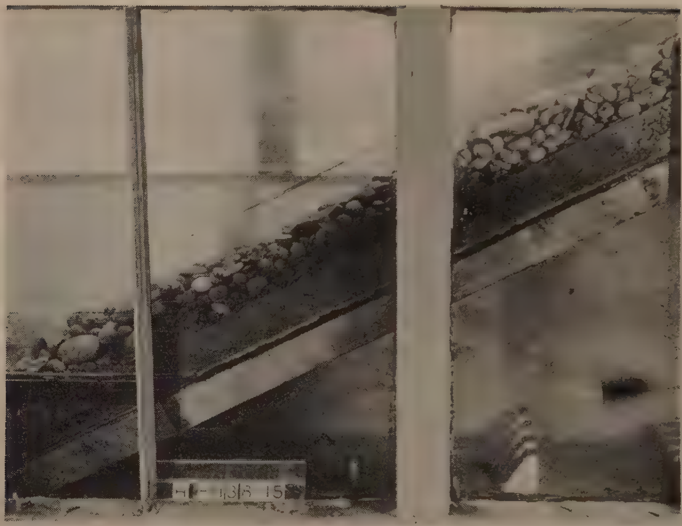
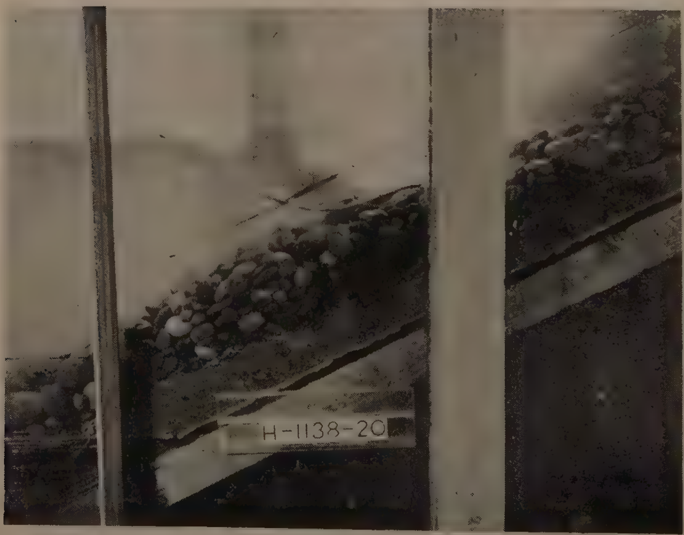


FIGURE 8. — GRAVEL MATERIAL SCREENED AT 3/4 INCH
BASE MATERIAL – FILTER CRITERIA

Figure 9



(a) Ready for operation



(b) After 23 hours of operation at 42 waves per minute--wave height 0.43 ft.

Yakima Project - Washington
KENNEWICK MAIN CANAL
TEST 2 (GRAVEL SEPARATED ON 3/4-INCH SCREEN)
Model study

3—Gravel Placed Dry and Raked

attempt was made to find a less expensive method of placing the cover rial and stabilizing it with a raking or harrowing process. It was thought vith harrowing the gravel material, the finer material would tend to set- the lower portion of the cover and the coarser material migrate toward urface; in other words, a separation with the coarse material on the sur- vas attempted to be produced by raking, Figure 10, rather than screen- s was done in Test 2.

e wave machine was started and operated at 32 waves per minute, with e height of 0.32 foot for a short time. This created only a small amount vement of the finer material. The wave machine was then stepped up to ves per minute, with a resulting wave height of 0.43 foot. The depth of was again held constant at 2.7 feet. As can be seen on Figure 10b, the wed material withstood the wave action much better than the unraked oist material placed in Test 1. However, beaching did start near the surface as the machine was first turned on, and a small amount of ing continued until the wave machine was turned off 24 hours later. photograph showing the model after it had been in operation for 19 hours be seen in Figure 10b. The final beach slope, after the model had been in tion for 24 hours, was approximately 5.14:1. This slope seemed to be r stable at the end of the 24-hour period.

4—Gravel Separated on 1-inch Screen

Test 4, the gravel was separated on a 1-inch screen, and 4 inches of rial smaller than 1-inch in size was placed on the 2:1 slope and covered 4 inches of the plus 1-inch material. The resulting analysis curve d with the base material, and filter criteria are shown in Figure 11. nodel was filled to a depth of 2.7 feet, turned on, and operated at 42 s per minute for approximately 45 minutes. Although the surface ma- withstood the 0.43-foot waves very well, some sand moved from the layer quite freely.

gure 12a shows a photograph of the model after 45 minutes of operation. rea from which the sand moved in the lower layer is seen in the photo- . At this condition, the surface material was very stable. To create a severe erosion condition, the water surface was then raised to a depth feet. The wave machine was operated at 42 waves per minute, and the ting waves increased from 0.43 foot to 0.5 foot from trough to crest. this larger wave, there was some movement of the surface material; ver, it was not considered serious. Increased movement of the sand the material smaller than 1-inch in size was noted.

study of the photograph in Figure 12b reveals that the sand had moved in wer layer to the full depth of 4 inches. It was obvious that the compara- larger voids in the plus 1-inch material allowed a flow of water through avel great enough to cause the fine sand to leach out through these voids. arger voids would not reduce the force of the water enough by the time it ed the fine sand material in the lower layer. The fine base material d under a protective blanket similar to that which was placed for Test 4 apparently leach out, as shown by the test.

Figure 10

(a) Rake used in Test 3

(b) After 19 hours of operation at 42 waves per minute--wave height 0.43 ft.

Yakima Project - Washington
KENNEWICK MAIN CANAL
TEST 3 (GRAVEL PLACED DRY AND RAKED)
Model study

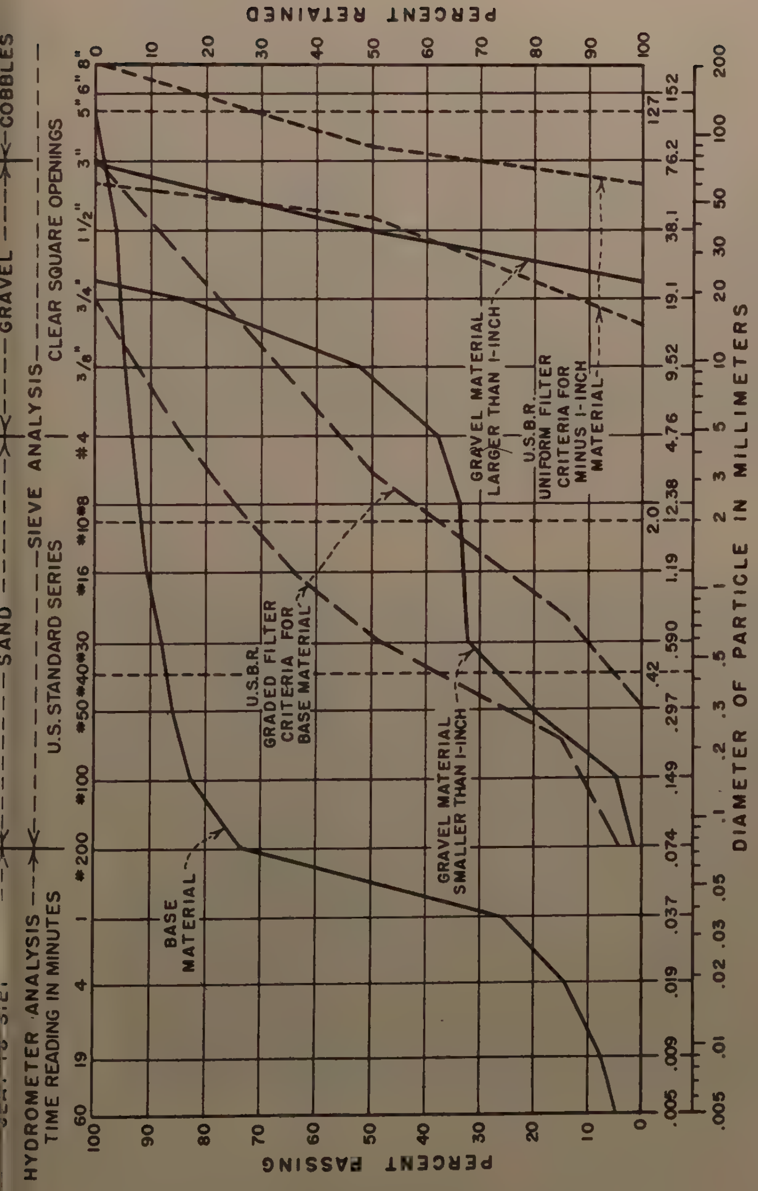
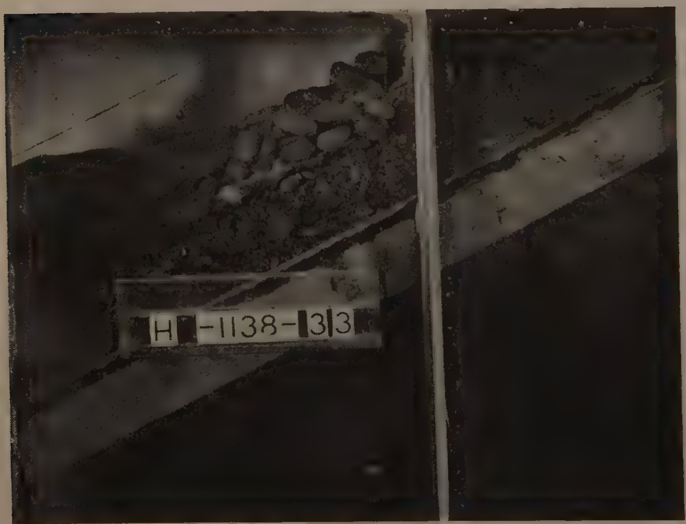


FIGURE 11.— GRAVEL MATERIAL SCREENED AT 1 INCH
BASE MATERIAL — FILTER CRITERIA

Figure 12

(a) After 45 minutes of operation--2.7 ft. depth--42 waves per minute--wave height 0.43 ft.



(b) After 20 more minutes of operation--3.2 ft. depth--42 waves per minute--wave height 0.5 ft.

Yakima Project - Washington
KENNEWICK MAIN CANAL
TEST 4 (GRAVEL SEPARATED ON 1-INCH SCREEN)
Model study

sts 5 and 6—Gravel Separated on 1/2-inch Screen

To find the optimum screening size, tests were continued, and in this test gravel was separated on a 1/2-inch screen. For Test 5, the resulting lysis curve with filter criteria and the bed material curve are shown in ure 13. Four inches of material smaller than 1/2-inch in size was placed the 2:1 slope and covered with 4 inches of material larger than 1/2-inch in e, Figure 14a. The material in this test was screened while damp, and it s noticed that the sand adhered to the coarse gravel particles even after siderable shaking in the screening machine. This same condition occurred Test 1. Water was again raised to a depth of 2.7 feet, and the wave machine s operated at 42 waves per minute, creating waves with a height of 0.43 foot. er a few waves had washed up the slope, a slide occurred. This slide ap- red to start in the area where the water from the waves flowed out of the vel material and continued to the toe of the slope. A photograph showing condition of the slope after the slide may be seen in Figure 14b.

Motion pictures taken of this test, including the slide, are included in the cord of the hydraulic model tests. From viewing the motion pictures, it peared that the sliding surface started at the surface of the plus 1/2-inch terial, progressed through the plus 1/2-inch material, then through the nus 1/2-inch material, and ended at the board surface at the toe of the pe. As the material had been placed in the same manner as for the other t, except that it was damp and the sand remained coated to the rock, it is ieved the sand reduced the friction between rocks in the top layer and the ce of the water flowing out of the blanket when a wave receded down the pe was enough to start the slide.

After the slippage, the material was removed, rescreened in a dry condi- n, and replaced in the same two layers, as for Test 5. The material placed this dry condition was called Test 6. The model was then backfilled to a oth of 2.7 feet, the wave machine was set as before and operated for 24 urs. No slippage occurred, and the fine material appeared stable; there s little movement of sand. Some of the surface material (larger than 1/2- h) moved, forming a beaching slope. This 1/2-inch and larger material l a tendency to move more readily than the 3/4-inch and larger or the 1- h and larger, which had been tested previously. The photograph, showing condition of the model after 23 hours of operation, is shown in Figure 15a.

sts 7 and 8

sts to determine effect of sand on slippage failure

Tests 7 and 8 were conducted in an attempt to determine the effect of the nd when it remained coated to the rock, causing failure. For Test 7, the avel material was screened while damp on the 3/4-inch screen and placed, in Test 2. A 4-inch layer of material smaller than 3/4-inch in size was vered by a 4-inch layer of material larger than 3/4-inch in size. The only ference between Tests 2 and 7 was that the material in Test 7 was screened mp, thereby leaving a coating of sand particles on the larger gravel parti- s. The flume was filled to a depth of 2.7 feet, and the wave machine was rted at 42 waves per minute, resulting in a wave height of 0.43 foot.

Immediately on starting the wave action, considerable slippage occurred. e slippage was less pronounced, however, than when the material was parated on the 1/2-inch screen. The coarse material slipped near the sur- e until it had decreased in thickness by approximately 1 inch. After

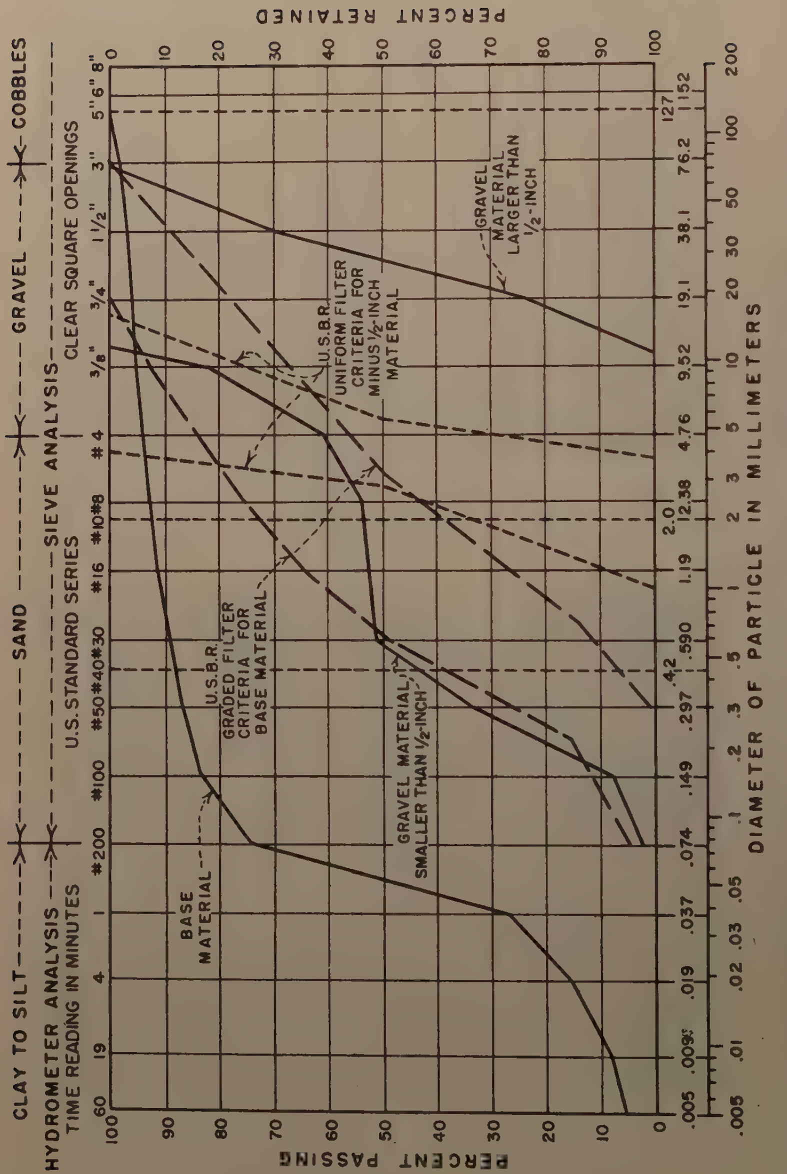


FIGURE 13.- GRAVEL MATERIAL SCREENED AT 1/2 INCH BASE MATERIAL - FILTER CRITERIA

Figure 14

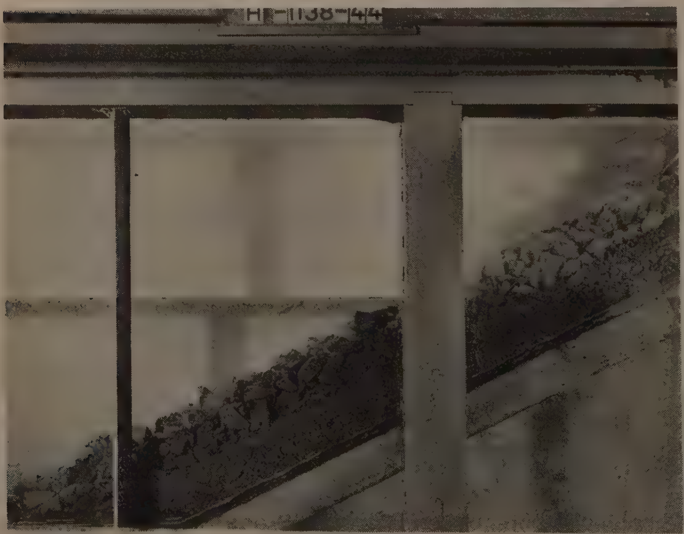
(a) Ready for operation

(b) Slippage that occured immediately after test started

Yakima Project - Washington
KENNEWICK MAIN CANAL
TEST 5 (GRAVEL SEPARATED ON 1/2-INCH SCREEN)
Model study

Figure 15

(a) Test 6--After 23 hours of operation
2.7 ft. depth--42 waves per minute--
wave height 0.43 ft.



(b) Test 9--Prepared for operation--
Recommended cover - talus
separated on 3/4-inch screen

Yakima Project - Washington
KENNEWICK MAIN CANAL
TESTS 6 and 9
Model study

roximately 1 minute of operation, the material then became stable. Fur-r operation produced little change. The test appeared to confirm the fact t the sand coated to large material contributed largely to the slippage ure in Test 5.

For Test 8, the gravel was placed, as in Test 1, well mixed, but instead of ng in a damp condition, it was dry. An 8-inch layer was placed on the 2:1 pe, the model was filled to a depth of 2.7 feet, and the wave machine rted at 42 waves per minute, creating 0.43-foot wave height. Approximately nches of beaching at the water surface occurred in the first 2 minutes, and er about 3-1/2 inches of beaching had occurred, the gravel appeared to be te stable. A photograph showing the condition of the cover after 5 hours of eration is shown in Figure 19. The dry gravel resulted in a more stable ndition than the damp gravel of Test 1.

st 10—Talus Material Moistened and Well Mixed

A 6-inch layer of talus material moistened and well mixed was placed on a slope in the testing flume. The method of placing was similar to Test 1, which gravel material was used. The flume was filled to a depth of 2.7 t, and the wave machine started at 32 waves per minute. There was move-nt of the finer talus material, but not as much as that observed in the grav-material of Test 1. After 20 minutes of operation, the frequency of the ves was increased to 42 per minute, and after an additional 20 minutes to per minute, with a resulting wave height of 0.32, 0.43, and 0.5 foot, respec-ely. At the higher frequency and wave height, there was considerable ero-n of the talus material which was washed down to form a beaching slope of 6:1. The model was operated for 2-1/3 hours at 62 waves per minute.

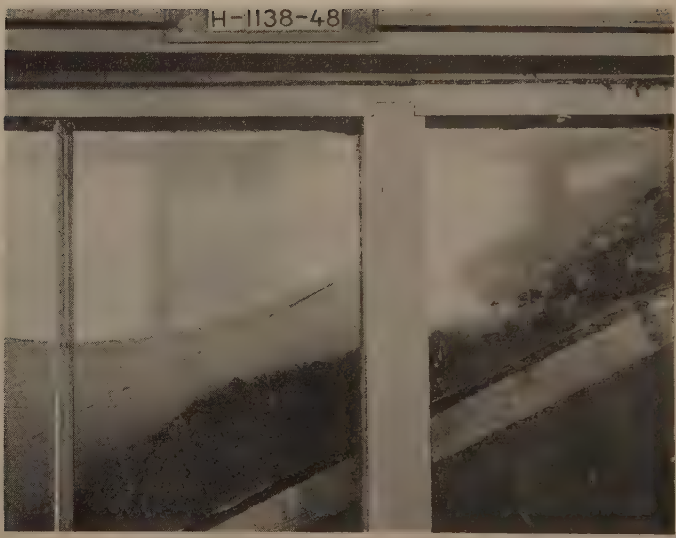
A photograph showing the condition of the talus material after 3 hours of al operation, described above, may be seen in Figure 16a. After the beach-slope had been recorded, the model was left running at 62 waves per nute, and the water was raised slowly to a depth of 3.2 feet, then lowered a depth of 2.2 feet. Raising and lowering the water surface caused various ve heights for 62 waves per minute. The scour observed showed consider-e material had moved down the slope, but at no point was the 2:1 slope ex-sed. A photograph showing the condition of the model after raising and vering the water surface is shown in Figure 16b. The model was operated a total of 5 hours. Test 10 indicated that the angular talus material was nsiderably more stable than the rounded gravel material placed in the me manner.

st 11—Talus Separated on 1/2-inch Screen

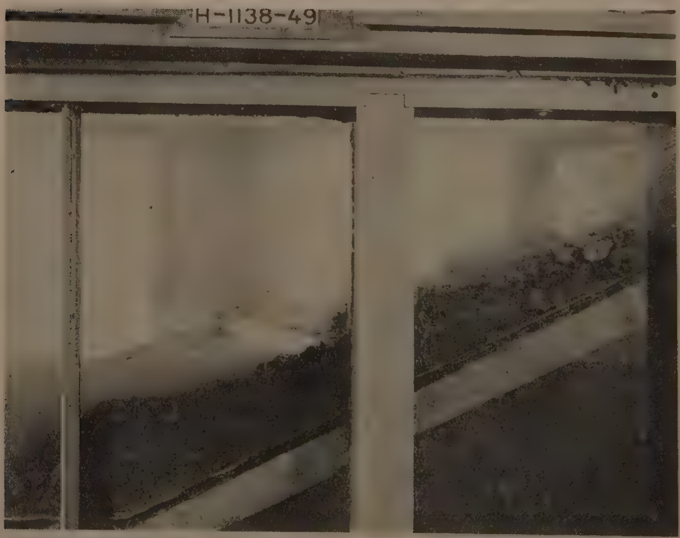
The talus material was dried and screened on the 1/2-inch screen, result-in the analysis curve shown in Figure 17. Four inches of the material aller than 1/2-inch in size was placed on the 2:1 slope and covered by 4 hes of material larger than 1/2-inch in size. The model was filled to a th of 2.7 feet, and the wave machine was started at 32 waves per minute, ing a wave height of 0.32 foot. At 32 waves per minute, there was very le movement of the surface material. The material in the bottom layers aller than 1/2-inch in size also appeared very stable.

As no damage occurred at 32 waves per minute, the frequency of the waves s increased to 42 and then to 62 waves per minute, and the model was rated for a 2-hour period. The resulting wave height was 0.43 and 0.5 foot,

Figure 16



(a) After 3 hours operation at various wave heights and frequencies



(b) After 5 hours operation with various waves and depths

Yakima Project - Washington
KENNEWICK MAIN CANAL
TEST 10 (TALUS MOISTENED AND WELL MIXED)
Model study

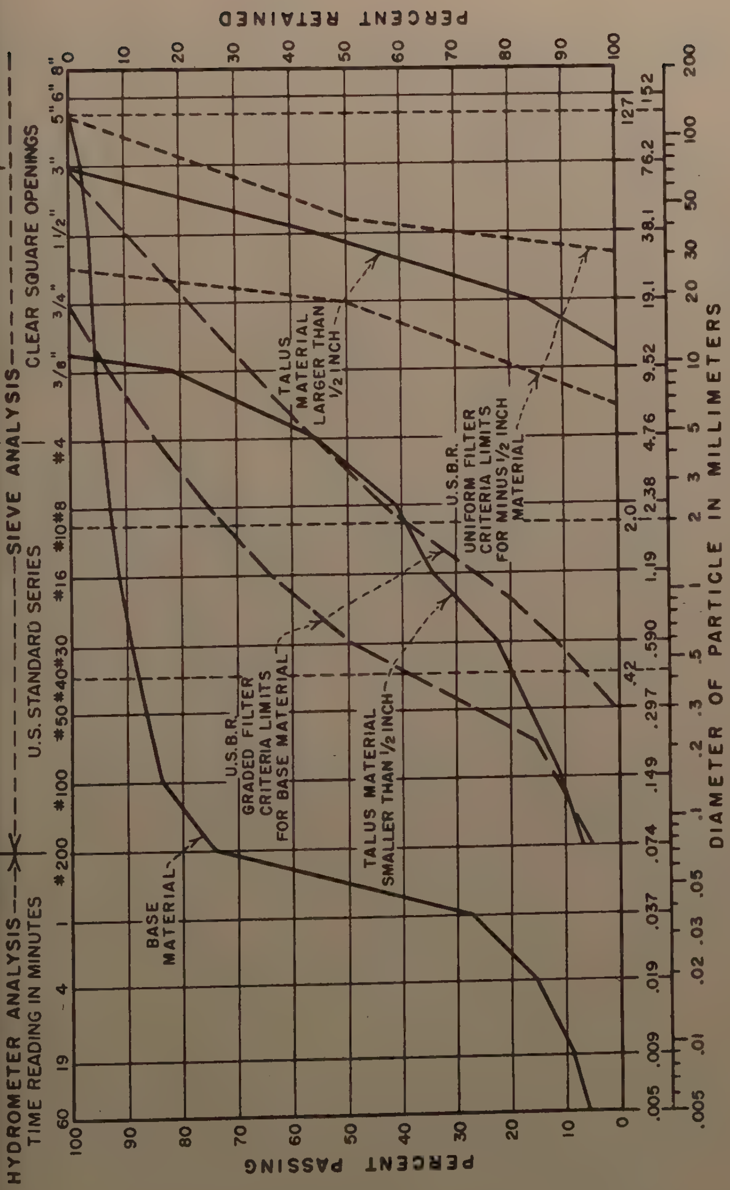


FIGURE 17.– TALUS MATERIAL SCREENED AT 1/2 INCH
BASE MATERIAL – FILTER CRITERIA

respectively. At the higher wave frequencies, the surface material moved more freely, and approximately 1-inch was eroded in forming the beaching slope. However, the material in the bottom layer below the 1/2-inch in size still appeared very stable and very little movement occurred.

Test 9—Recommended Cover—Talus Separated on 3/4-inch Screen

The mixed talus material was screened on the 3/4-inch screen, together with the base material analysis curve and the Bureau of Reclamation filter criteria. The resulting analysis curve is shown in Figure 18. Before placin the talus material on the 2:1 slope, 3.6 percent moisture, by dry weight, was added to the material smaller than 3/4-inch in size to simulate field conditions. Four inches of the material smaller than 3/4-inch in size was placed on the 2:1 slope and covered by 4 inches of material larger than the 3/4-incl size, Figure 15b. The model was filled to a depth of 2.7 feet, and the wave machine was started. The model was operated at a frequency of 32 waves per minute, creating 0.32-foot waves for 15 minutes. As the material appeared to be very stable, the frequency of the wave machine was increased t 42 waves per minute. The model at this frequency and wave height (0.43 foo was operated for 30 minutes. There was a small amount of movement of the fine material. The wave frequency was then increased to 68 waves per minute, with a resulting wave height of 0.5 foot.

The 0.5-foot wave approaching the slope may be seen in the photograph, Figure 20a. At 68 waves per minute, the surface material was still relative stable. There was a little movement of the fine material in the lower layer; however, this movement in the lower layer appeared to stop after about 2 hours of operation. The model was operated at 68 waves per minute for approximately 5 hours. The photograph, shown in Figure 20b, gives the condition of the slope at the end of the 5-hour period. A comparison of Figures 1 and 20b shows very little change in the appearance of the material.

Test 12 (Fine Base Material)

A 6-inch layer of the fine base material (analysis curve shown in Figure was subjected to wave action. In preparing the material, enough water was added to bring the moisture content up to 18 percent of the dry weight of the material. The 18 percent moisture is the optimum moisture content for maximum compaction. The material was placed on the 2:1 slope in layers c approximately 2 inches, and each layer was tamped to near maximum compaction with a wooden tamper. Before placing the base material on the 2:1 board slope, it was roughened with a layer of metal lath to prevent any tendency for the base material to slide on the plywood board slope.

The water was raised slowly in the flume to a depth of 2.7 feet, Figure 2 There was some settlement of the material on the slope, and near the water surface, cracks could be observed. The wave machine was started and operated at 32 waves per minute. The wave height varied from 0.25 to 0.40 foot as the material washed down the slope. During this operation, complet failure of the base material slope occurred. Immediately on starting the wave machine, the earth material began to fail. The photograph taken 2 minutes after operation started may be seen in Figure 21b. The density cu rent of fine material flowed down the slope to the bottom of the flume. Afte 12 minutes of operation, the 2:1 slope was completely exposed. The model was operated for 1 hour, after which it was turned off and slowly drained.

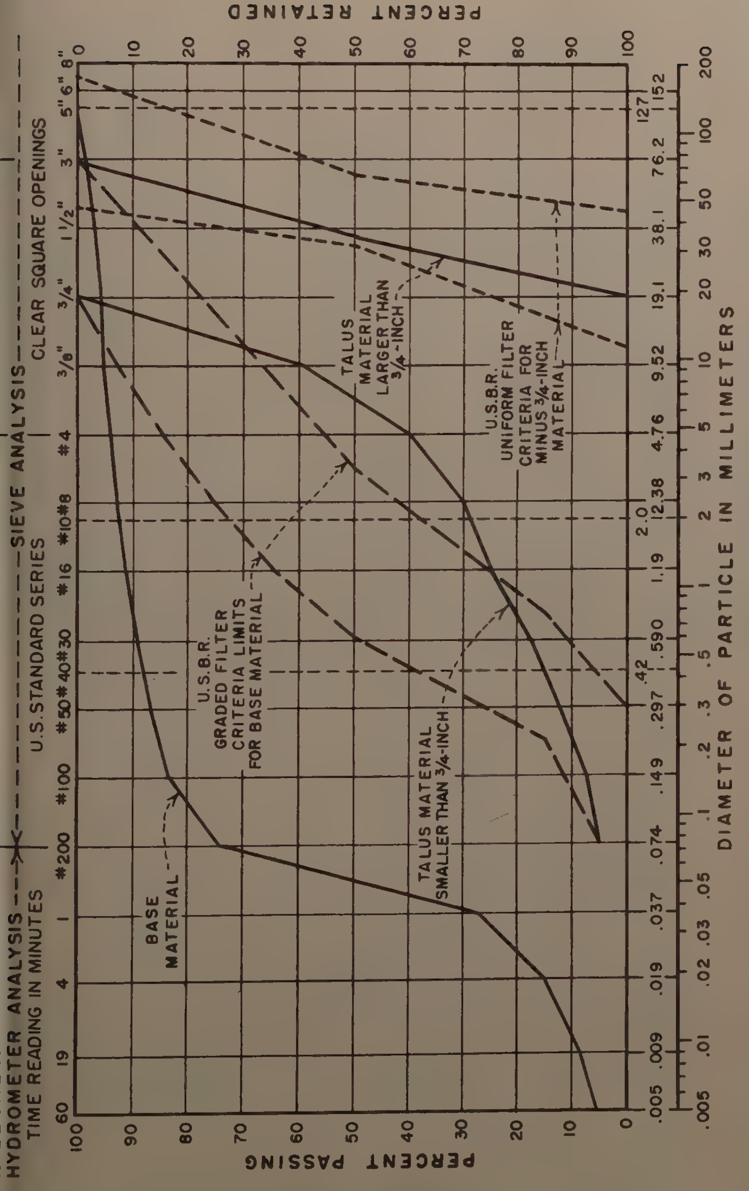


FIGURE 18.—TALUS MATERIAL SCREENED AT 3/4 INCH — RECOMMENDED FOR COVER BLANKET BASE MATERIAL — FILTER CRITERIA

Figure 19
Yakima Project - Washington
KENNEWICK MAIN CANAL
TEST 8 - AFTER 5 HOURS OF OPERATION
Model study

The fine base material assumed an approximately level position, as shown Figure 22a.

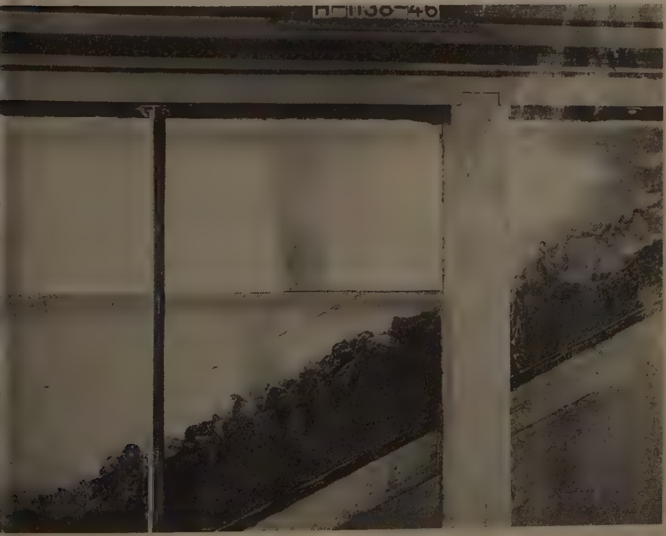
Test 13—Base Material and Plus 1/2-inch Talus Material

A 4-inch layer of the base material was compacted on the 2:1 slope and covered with 4 inches of talus material larger than 1/2-inch in size, Figur 17. The fine base material was placed at optimum moisture content (18 pe cent by dry weight) and compacted in thin layers with a wooden tamper. T talus material was dumped on the compacted base material, simulating the way that it would possibly be placed in the field, Figure 22b. The model w slowly filled to a depth of 2.7 feet, and the wave machine was started at 32 waves per minute, giving a wave height of 0.32 foot.

Immediately after the model was started, the fine base material began leaching through the talus cover, the material being leached out formed a

Figure 20

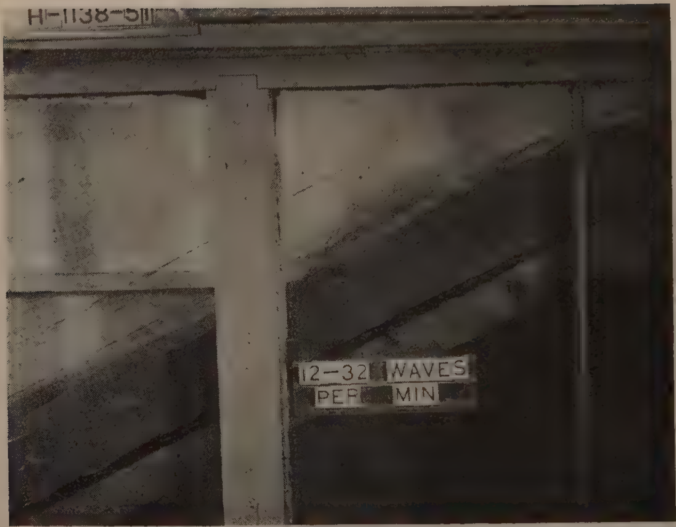
(a) A 0.5 ft. wave approaching the 2:1 slope



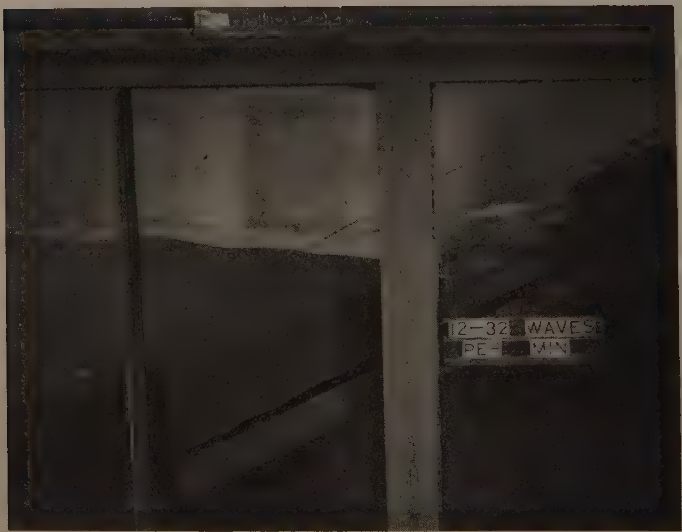
(b) Slope after 5 hours of operation at varying wave heights and frequencies

Yakima Project - Washington
KENNEWICK MAIN CANAL
TEST 9 (RECOMMENDED COVER -
TALUS SEPARATED ON 3/4-INCH SCREEN)
Model study

Figure 21



(a) Ready for operation

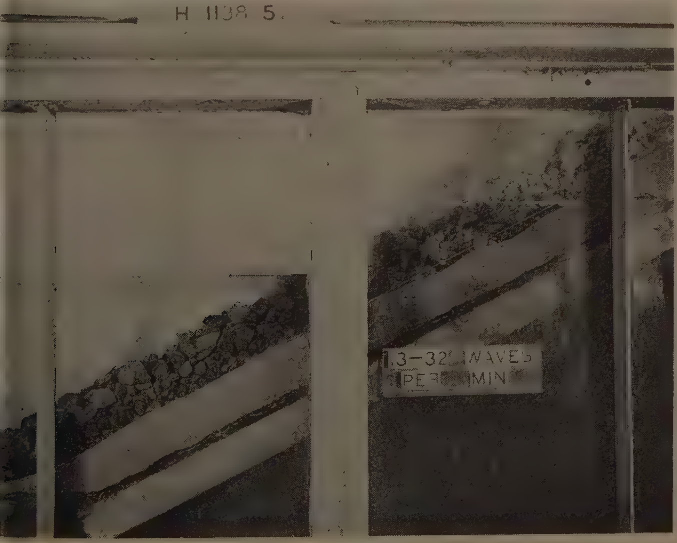


(b) Two minutes after operation started at 2.7 ft. depth and 32 waves per minute--wave height 0.26 ft.

Yakima Project - Washington
KENNEWICK MAIN CANAL
TEST 12 (FINE BASE MATERIAL)
Model study

Figure 22

(a) Test 12--after 1 hour of operation at 2.7 ft. depth--32 waves per minute--wave height 0.26 ft.



(b) Test 13--ready for operation. Base material covered with plus 1/2-inch talus

Yakima Project - Washington
KENNEWICK MAIN CANAL
TESTS 12 and 13
Model study

density current which moved down the slope in the water immediately above the talus cover and continued on to the flume floor. The density current ma be seen in the photograph of Figure 23a which was taken 7 minutes after the model was turned on. After 30 minutes of operation at 32 waves per minute the wave machine frequency was increased to 42 waves per minute. The increase in frequency increased the wave height to 0.43 foot. At this increase frequency, the leaching of base material increased.

A photograph taken 90 minutes after operation started, shows the base material had washed within approximately 1/2 inch of the board slope (Figu 23b). The model was operated an additional 17-1/2 hours at 42 waves per minute. During this time, the upper part of the slope failed completely. Th finer material was completely washed from beneath the talus cover. The photograph showing this failure may be seen in Figure 24a.

Test 14—Recommended Cover and Base Material

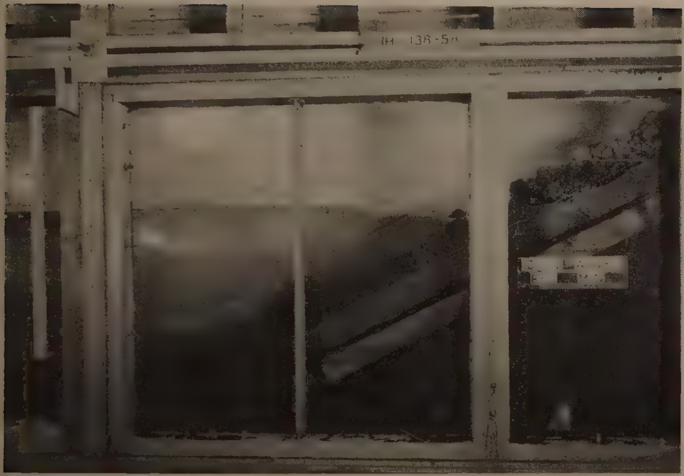
A final test was run using the talus material as protective cover for the fine base material. The moisture in the base material was increased to optimum moisture content, and a layer 4 inches thick was placed by the tampi operation explained previously. The talus material was screened on the 3/ inch screen. Four inches of talus material smaller than 3/4-inch in size w placed by dumping over the fine base material, and on top was placed 4 inch of talus material larger than 3/4-inch in size, Figure 24b. The analysis curves are shown in Figure 18. The total thickness of the test section was 12 inches. The model was filled to a depth of 2.7 feet and operated for 28 hours at a frequency of 42 waves a minute, giving a wave height of 0.43 foot

As shown by Figure 25, the test section withstood the wave very well, ar very little movement occurred in the talus material. No movement occurre in the fine base material. To check the stability of the section, the model v operated for a total of 128 hours. After the first 28-hour period, the operation was as follows: 24 hours at 3.2 foot depth, 42 waves per minute, 0.43 foot wave height; 24 hours at 2.2 foot depth, 42 waves per minute, 0.42 foot wave height; 4 hours at 2.7 foot depth, 22 waves per minute, 0.6 foot wave height; 24 hours at 2.7 foot depth, 51 waves per minute, 0.62 foot wave heig and 24 hours at 3.03 foot depth, 22 waves per minute, 0.6 foot wave height.

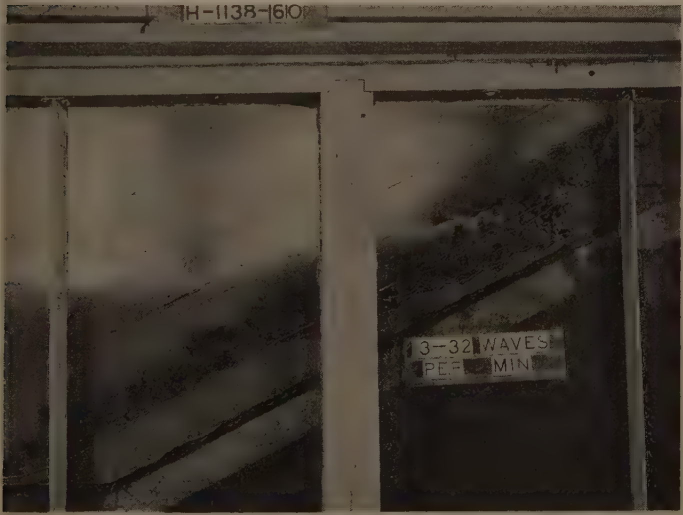
As may be seen from a study of Figure 26a, the material withstood the operation very well. At the frequency of 51 waves per minute, there was some movement of the surface material. During the complete test, the talu material below 3/4-inch in size was very stable, and no leaching of the fine base material was observed. The waves of 22 per minute frequency ran up and down the slope from maximum to minimum, a distance of 2.2 feet. The 0.6-foot high waves at 22 waves per minute resulted in the most damaging condition where leaching action was concerned. However, no leaching of th base material occurred.

The section was operated to failure after the above test, the water surfa was raised to a depth of 3.03 feet, and the wave machine set for 45 waves minute, with a resulting wave height of 0.75 foot. For this condition, the m terial failed. Failure started on the surface and progressed to the bed ma terial. Total failure occurred in 12 minutes, as may be seen from the pho graph of Figure 26b. The large waves were large enough to wash the large gravel particles off the slope, exposing the finer gravel and eventually the fine base material.

Figure 23



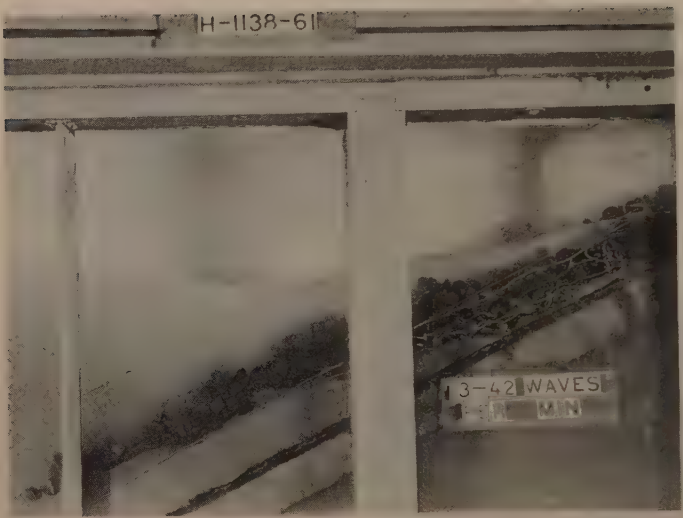
(a) After 7 minutes of operation at 32 waves per minute--depth 2.7 ft.--wave height 0.26 ft.



(b) After 90 minutes of operation at 32 and 42 waves per minute leaching of base material progressed as shown by lines on the photograph

Yakima Project - Washington
KENNEWICK MAIN CANAL
TEST 13 (BASE MATERIAL AND
PLUS 1/2-INCH TALUS MATERIAL)
Model study

Figure 24



(a) Test 13--after the model was left running all night at 42 waves per minute. Base material completely leached out in area near water surface

(b) Test 14--ready for operation. Base material covered with talus separated on 3/4-inch screen - recommended cover

Yakima Project - Washington
KENNEWICK MAIN CANAL
TESTS 13 and 14
Model study

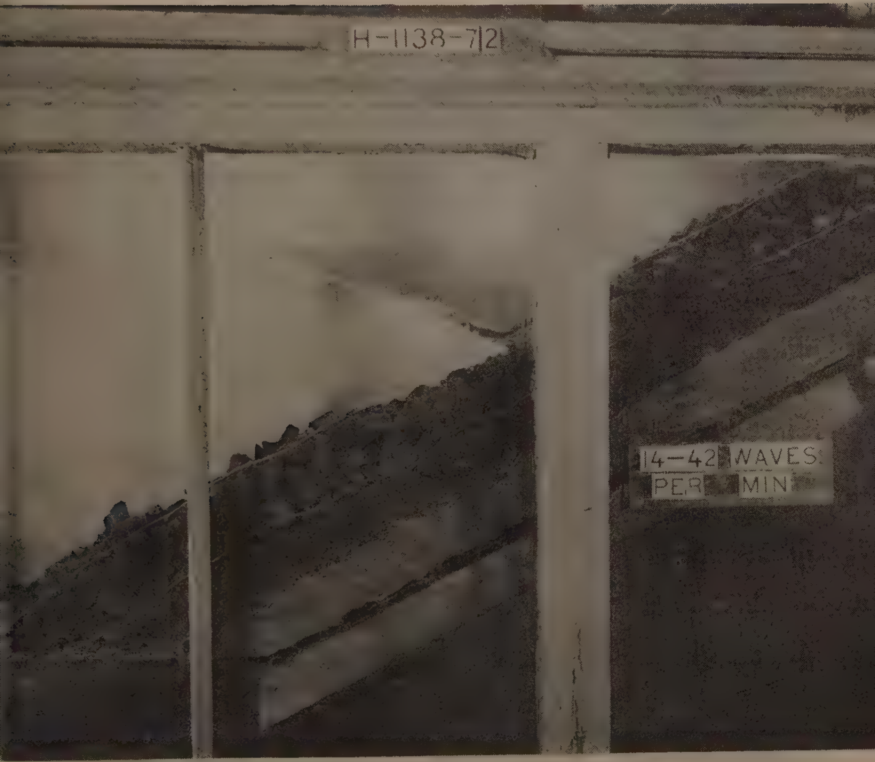


Figure 25

Yakima Project - Washington
KENNEWICK MAIN CANAL
TEST 14 (RECOMMENDED COVER) AFTER 28 HOURS OF OPERATION
AT 42 WAVES PER MINUTE--2.7 FT. DEPTH--WAVE HEIGHT 0.43 FT.
Model study

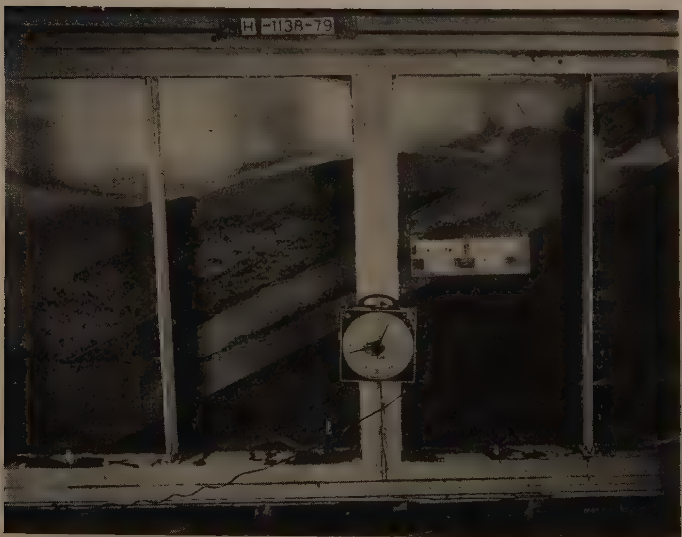
Results and Conclusions of Hydraulic Model Tests

The gravel used in the first test was well rounded, and sand adhered to the ırgest particles when the material was moistened and mixed. When sand ad-ered to the larger particles, failure of the 2:1 slope by a slip occurred. The ounded particles of the gravel moved more readily when exposed to wave ction than did the same mean size angular particles of the talus material sed in Tests 9, 10, 11, 13, and 14.

The tests indicated it was desirable to screen the cover material and place in two layers, the larger material protecting the finer material. The larger e average sized material on the surface layer, the more stable was the sur-ace to wave action and the more susceptible were the fines to leaching. The maller the average size of the surface material, the more susceptible the urface was to movement by wave action.

Figure 26

(a) After 128 hours of operation at various depths, wave heights and frequencies



(b) Failure of slope from waves 0.75 ft. high and 45 per minute--approximately 12 minutes after final test started

Yakima Project - Washington
KENNEWICK MAIN CANAL
TEST 14 (RECOMMENDED COVER)
Model study

The material that fell within the Bureau of Reclamation filter criteria mits appeared to provide an effective filter against leaching of fine materi-s by wave action. The study indicated that a protective material must be ne enough to prevent leaching of the base material through it and that par-cles on the exposed surface of the protective material must be of sufficient ze and weight to resist rolling down the slope from wave action.

The study further indicated that the angular talus particles form a more able protection blanket than the rounded gravel particles.

The results of Tests 9 and 14 showed the best protective cover was the lus material separated on a 3/4-inch screen and placed in 2 layers: (1) a yer of talus material smaller than 3/4 inch placed over the fine base ma-rial, and (2) a layer of talus material larger than 3/4 inch placed on the ırface. The resulting analysis curves in the Bureau of Reclamation filter riteria are shown in Figure 18. The test showed that the layers of cover aterial should not be less than 4 inches in thickness for adequate resistance erosion by small waves.

ACKNOWLEDGMENT

The hydraulic model tests were conducted in the Hydraulic Laboratory the Bureau of Reclamation, Denver, Colorado. Most of the tests were made P. F. Enger of the Sediment Investigations Group. The tests were per-rmed for the Diversion Works Section, Canals Branch of the Bureau of eclamation.

REFERENCES

"The Use of Laboratory Tests to Develop Design Criteria For Protective Filters" by K. P. Karpoff, Proceedings ASTM, Volume 55, 1955. Also issued as Bureau of Reclamation Earth Laboratory Report No. EM-425.

"Soil Mechanics in Engineering Practice" by Karl Terzaghi and Ralph B. Peck, John Wiley and Sons, Incorporated, 1948.

"Land Drainage and Flood Protection" by Bernard A. Etcheverry, McGraw-Hill Book Company, 1931.

"Ground Water, Its Development, Uses and Conservation" by E. W. Bennison, Edward E. Johnson Company, Incorporated, 1947.

"Flood-Erosion Protection for Highway Fills" by C. J. Posey, Member ASCE, Paper No. 783, Volume 81, Proceedings ASCE, August 1955.

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

HYDRAULIC MODELS OF THE ST. LAWRENCE POWER PROJECT

John B. Bryce[1]

ABSTRACT

Hydraulic models were used extensively in the design of many hydraulic eatures of the St. Lawrence Power Project and in the development of a plan or controlling the river during construction. A description is given of the roject and the models, and some of the more important findings are disissed.

INTRODUCTION

In the design and construction of the hydraulic features of the St. Lawrence ower Project, a fundamental part was played by hydraulic models. In all, even models were constructed for the Power Project, some being compreensive river models and others reproducing complete structures or parts of ructures. The models were constructed and tested at the Hydraulic Model aboratory of the Hydro-Electric Power Commission of Ontario for the use the Commission and the Power Authority of the State of New York, who ere jointly responsible for the design and construction of the Power Project. the models, the final detailed design of all channel enlargements in the ternational Rapids section was developed. Also evolved and demonstrated as a satisfactory plan of river control which would permit the construction the channel enlargements and the structures without adversely affecting ver levels and flows, and without interfering with existing navigation. In adtion, the optimum location of temporary and permanent structures was indiited, and assistance was given in predicting flow and velocity conditions durg construction. The hydraulic design of certain features of the structures as also developed. In the Power Project Model investigation, many interestg and complex problems were encountered. It is the purpose of this paper

te: Discussion open until October 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2022 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of CivilEngineers, Vol. 85, No. HY 5, May, 1959.

Hydr. Engr., The Hydro-Electric Power Commission of Ontario, Toronto, Canada.

to describe the models, and to outline the major design problems and the methods used in their solution.

Brief Description of Project

As shown in Fig. 1, the St. Lawrence Power Project extends throughout the International Rapids section of the St. Lawrence river from Ogdensburg, New York, and Prescott, Ontario, downstream to Cornwall, Ontario, a distance of about 45 miles. In nature a fall of approximately 92 feet occurred in this distance, the main concentrations being at: the Galop Rapids, 9 feet; the Plat Rapids at Ogden Island, 12 feet; and the Long Sault Rapids, 30 feet; the remainder of the fall occurring at lesser concentrations and as river slope. The flow in the International Rapids Section is remarkably uniform as a result of the natural regulating effect of the Great Lakes. The long term average flow is slightly greater than 240,000 cfs, while the maximum and minimum monthly mean recorded flows between 1860 and 1954 were 315,000 cfs and 166,000 cfs respectively.

The principal features of the Power Project may be briefly described and summarized as follows:

The Powerhouse

Located at the outlet of the channel around the north side of Barnhart Island, a short distance upstream from Cornwall, the powerhouse is some 3,120 feet long, about 160 feet high and contains 32 generating units with a total rated capacity of about 2,500,000 hp at 81.0 feet head. Six ice sluices also are provided. Together with the Long Sault Dam and 13 miles of dykes, this power dam enables the level to be raised over 80 feet at this point. Downstream from the powerhouse to Polly's Gut, channel enlargements are being undertaken to lower tailwater levels.

The Long Sault Dam

The Long Sault Dam is required to contain the power pool by closing the main river channel south of Barnhart Island. This dam provides the spillway for the project and must be capable of discharging the full flow of the river. Thirty sluices, each 50 feet wide, are provided, and each with a gate 52 feet wide and 30 feet high. The dam is about 2,400 feet long and is about 110 feet high at the deepest point.

The Massena Intake

An intake structure is provided near the entrance to the Massena Power Canal to permit continuance of the flow to the Aluminum Company plant during construction of the project and domestic supply thereafter. The intake contains four sluices or ports each controlled by a gate.

Iroquois Control Dam

Located just upstream of Iroquois, Ontario, the Iroquois Control Dam enables the St. Lawrence River flows to be controlled and regulated. The dam is some 2,340 feet long, 70 feet high, and contains 32 sluices, each 50 feet wide, and each provided with a gate 52 feet wide and 48 feet high.

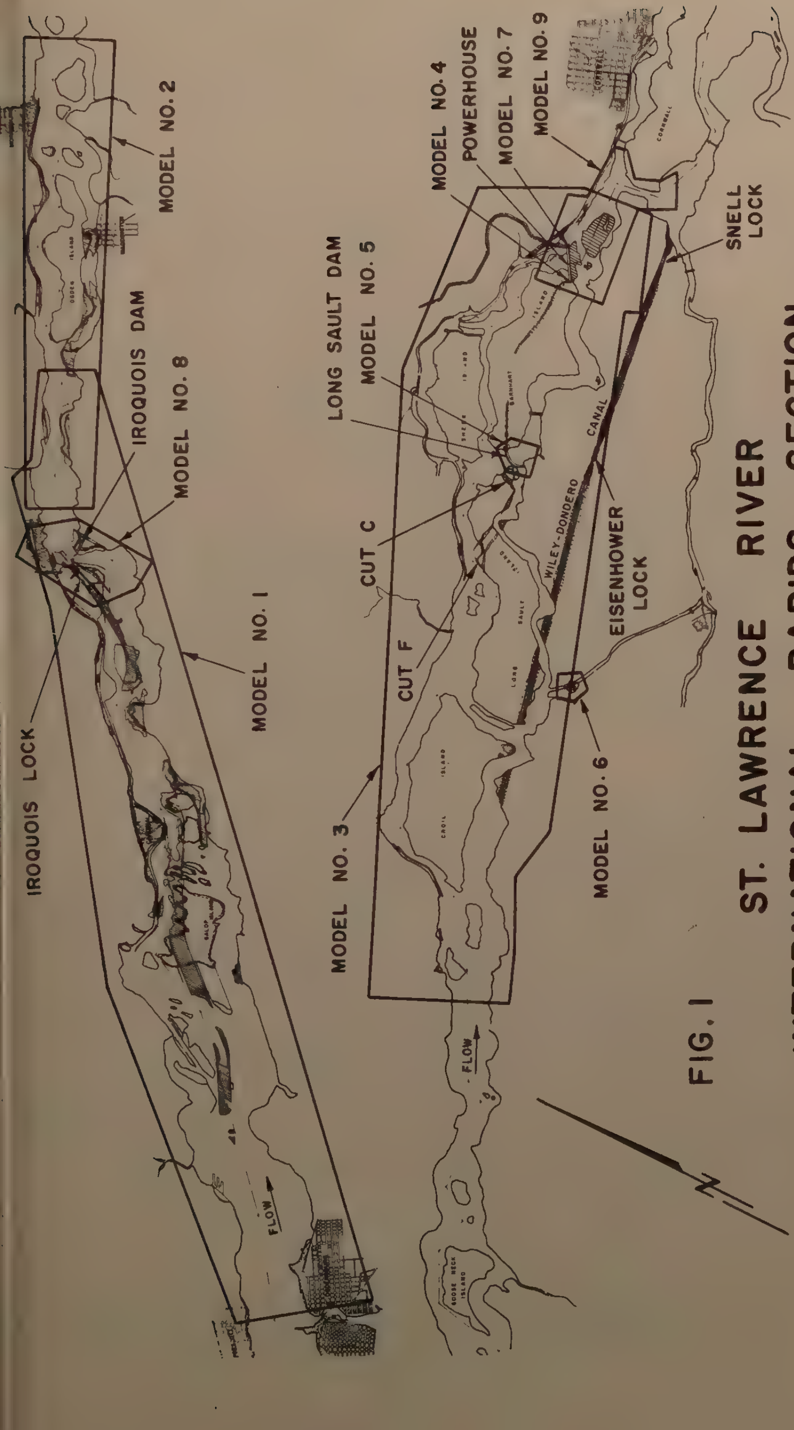


FIG. I ST. LAWRENCE RIVER INTERNATIONAL RAPIDS SECTION LOCATION AND EXTENT OF MODELS

Channel Enlargements Morrisburg to Sparrowhawk Point

Extensive channel enlargements are required in this reach of the river to provide velocities not exceeding those at which an ice cover may form by packing during winter operation. These enlargements also must provide an adequate course for 27-foot navigation.

Channel Enlargements Cardinal to Chimney Point

In this reach, very extensive channel enlargements are required to provide a deep-draught navigation channel and velocities not exceeding those stipulated for navigation.

Navigation Features

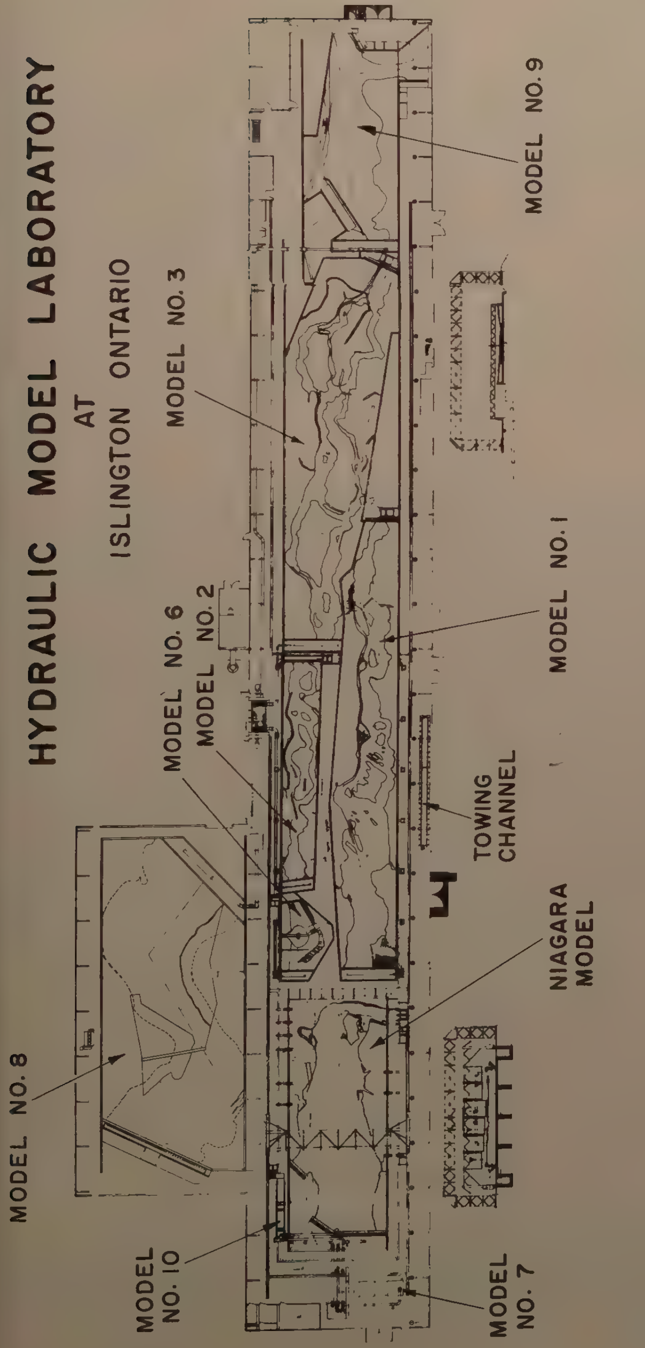
While not directly connected with the Power Project, certain navigation features in the International Rapids section are interrelated and will be mentioned here. The Canadian St. Lawrence Seaway Authority has improved the channel north of Cornwall Island and portions of the channel south of Cornwall Island. The United States Seaway Development Corporation has enlarged the remainder of the south channel around Cornwall Island and has constructed the Snell Lock, the Eisenhower Lock and the Wiley-Dondero Canal upstream from the locks. Twenty-seven-foot navigation will enter the headwater pool by this means and after leaving the Wiley-Dondero Canal will sail up the flooded river channel to Morrisburg and the enlarged river channel to Iroquois. At Iroquois, the St. Lawrence Seaway Authority has provided a lock to by-pass the Iroquois Dam. After passing through the lock, shipping will proceed up the enlarged channel to Chimney Point and the natural river therefrom. The location of these navigation features may be noted on Fig. 1.

Scope of Model Investigation

The hydraulic problems associated with the St. Lawrence Power Project were many and varied, and all were accentuated by the large scale of the Project and the continuously large flow in the river. For many of these problems it was considered that the construction and testing of hydraulic models would yield the best, and in some cases the only reliable method for their satisfactory solution. Accordingly, it was decided to carry out a comprehensive program of model testing to provide the maximum assurance that the Project would be constructed and would operate as planned. Also from past experience, it was considered probable that substantial economies might be realized in view of the large scale of the project. Construction of the first St. Lawrence models commenced in 1953 and testing began later that year. From that time throughout the construction of the project building and testing of the models proceeded continuously. As final designs were completed, the emphasis shifted to the steps in construction and the provision of information useful to construction.

In general, the problems to be investigated fell into four categories:

(1) The design of channel enlargements.
(2) Problems at structures, such as their optimum location, the design of their hydraulic elements, and the provision of adequate discharge capacity both during and after construction.



GENERAL ARRANGEMENT OF MODELS

0 50 100
SCALE - FEET

FIG. 2

(3) The development of a plan of river control during the construction of the Project.
(4) The determination of velocities and currents in the navigation channels throughout the Project.

For the investigation of the problems eleven hydraulic models were constructed. Four of these models were distorted-scale river models and reproduced altogether over 40 miles of the St. Lawrence River. The remainder were natural scale models, three of these being large comprehensive models of principal structures. With regard to the large river models, it might be observed that one of the principal reasons for their construction was that in the International Rapids section many islands divided the flow and abrupt changes in direction occurred at some critical points. It was considered that by no other means could reliable information be obtained under flooded conditions of the flow distribution through the multiple channels, the velocity distribution across the sections, the actual velocity in the navigation channels, and the detailed effects of modification in channel enlargements.

Collection of River Information

To reproduce over 40 miles of the St. Lawrence River in the models, it was essential that accurate river bed contours and topographic data be obtained for their construction, and that river levels, flow distributions and currents be available for model verification. The collection of such data over the length of such a large river posed a major problem.

Fortunately, earlier surveys of the International Rapids section provided a general coverage of subaqueous contours and other topographic data and these were used as a base. To verify the information, echo soundings were carried out at sections about one mile apart over the length of the river to be reproduced in the models. Where discrepancies occurred, or in critical areas, extensive programs of additional echo soundings were carried out. In the Long Sault Rapids section, soundings were made from helicopters as the most feasible method. Aerial photographs of the whole reach were made to provide accurate mapping.

With regard to hydrometric information, records from several long term lock gauges were available, and these were augmented by over 30 automatic recording gauges which provided a more general coverage. However, for verification purposes a much denser coverage was required and this was obtained by establishing over 100 temporary gauges. Intensive reading of these gauges at selected flows provided the necessary information. Fortunately, these gauges were established at the time of the peak flow in 1952, and as the flow subsequently subsided a considerable range in levels was obtained. Current meterings to determine velocities and flow distribution around the islands were made, and float observations indicated the current pattern.

Islington Model Testing Laboratory

All the models for the St. Lawrence Power Project were constructed and tested at Ontario Hydro's Hydraulic Model Laboratory at Islington, just west of Toronto, Ontario. This laboratory, constructed specifically for model testing, was begun originally for Niagara redevelopment model tests and was

ended for the St. Lawrence investigation. The building is about 500 feet g and 70 feet wide over above three-quarters of its length. Over the re- ining one-quarter, the width is doubled to 140 feet, as shown on Fig. 2, ich also indicates the plan of the laboratory and the general arrangement the models. The laboratory is serviced by a travelling gantry crane over st of its length, and accurately levelled rails provide vertical control from velling point gauges. Two constant-level head tanks provide the flow for e models through distribution mains; and sumps generally encircling the ilding provide the return for recirculation. A special use of Bailey bridging aterial provides uninterrupted floor space throughout.

The River Models

The four river models constituted a large and important part of the St. wrence model test investigation. As these models have many features in mmon, they will be described and discussed collectively rather than indi- dually.

escription of River Models

In Fig. 1 is shown a plan of the International Rapids section on which is arked the limits of the individual models. Model No. I extends from gdensburg to Leishman's Point and reproduces about 16 miles of the river. is model contains the Galop rapids reach and the Sparrowhawk Point— oquois reach where very large channel enlargements were required, and in- udes also the Iroquois Dam and the Iroquois 27-foot navigation lock. Model). 2 reproduces approximately 8 miles of the river from above Point Three oints to below Canada Island and contains the Ogden Island reach where im- rtant channel enlargements were necessary. Model No. 3 extends from ove Croil Island to Cornwall, a distance of about 14 miles, and contains the ong Sault Dam and Power House. Model No. 3 was used for the dewatering udies of the Long Sault Dam, and it was considered necessary to extend the odel upstream beyond Croil Island where the river is in a single channel in der to obtain the right flow distribution in the multiple channels in the Long ult area. In Fig. 3 is shown a photograph of Model No. 3 which is typical of e various river models constructed. Model No. 9, reproduces the area from ove the Powerhouse to the upper end of Cornwall Island and includes the ilrace area where channel enlargements are necessary to obtain an eco- mic head regain at the plant. It will be noted that these four river models produce the entire International Rapids reach from Ogdensburg to just above ornwall, with the exception of an 8 mile reach in the Goose Neck Island area here problems warranting model study were not considered to exist. It was nsidered desirable to construct several models rather than one long con- uous model, as work could proceed simultaneously on all models without e tests on one reach interfering with tests on other reaches.

oice of Model Scales

The scales chosen for river models 1, 2 and 3 were 1:500 horizontally and 100 vertically. The choice of the horizontal scale ratio was largely dictated laboratory space requirements consistent with other requirements. A reful study was made of the optimum placement of the models in the

Fig. 3. River Model No. 3 showing Powerhouse and Dykes in Foreground

laboratory, and as will be seen from Fig. 2, a larger horizontal scale would have required a very considerable increase in laboratory space. This scale was also considered satisfactory from accuracy and workability standpoints.

Due to the horizontal scale selected and the large river width relative to its depth, distortion of the vertical scale was considered necessary for simi- lar performance of model and prototype. The measurement of model veloci- ties and depths was of great importance in these models, and it was desired that the vertical scale be sufficiently large to enable these quantities to be measured with the required precision. However, as the vertical scale in- creased, the difficulty of producing the required model roughness also in- creased. A distortion ratio of 5:1 was selected, thus producing a vertical scale of 1:100, which was considered the largest feasible in view of the roug ness requirements, and adequate from the measurement standpoint. From past experience this distortion ratio had proven successful in other models a similar nature. The scale distortion in these models required also that th Iroquois Dam and the Long Sault dam structures be distorted as well. In tes where distortion of these structures affected the results, their true head- discharge relationship was determined from the larger natural scale models of these structures, and the correct levels produced in the distorted-scale models.

The scales chosen for Model No. 9 of the power house tailrace were 1:16 horizontally and 1:80 vertically. It was considered that for this model the scales should be enlarged and the distortion reduced, as a detailed economic

dy was desired and as a considerable change in direction of the flow oc-
rred on entering the tailrace.

nstruction of River Models

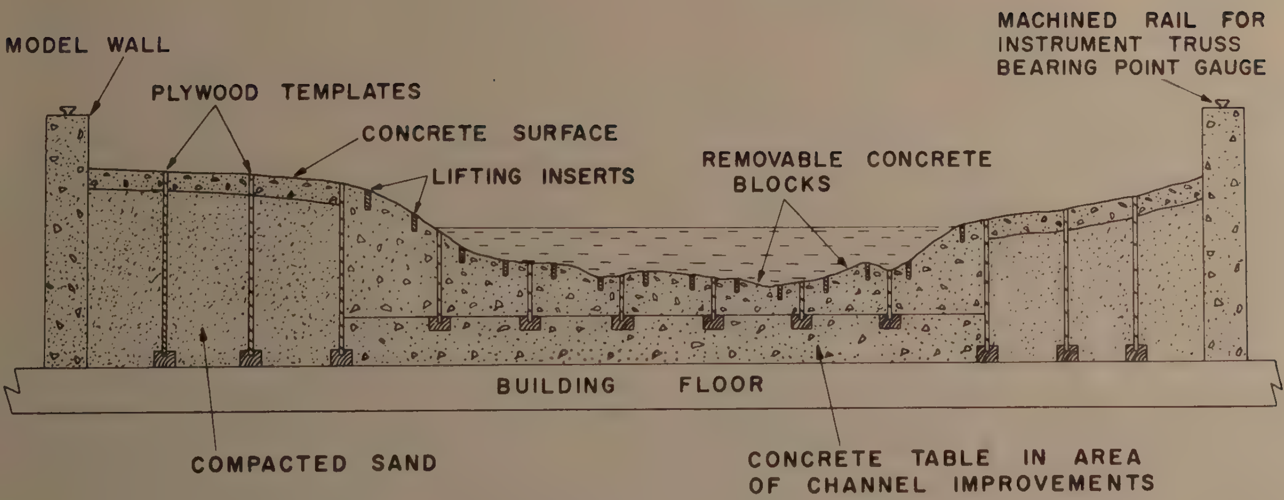
The river models were all constructed in the same manner using a method ich has been found to be satisfactory. Over the entire model area, thin wood templates were erected on a two-foot grid running both laterally and gitudinally. The tops of the templates were accurately cut to the surface vation of the ground or river bed. In the areas where no channel im- vements were expected, sand was placed between the templates to within inches of their tops, and then a layer of concrete was placed on top of the d to produce the final surface. Between templates, a "router" on a travel- point gauge traced the contours to the correct elevation. In areas where nnel changes were expected a different procedure was used. A concrete ble" was poured just below the minimum grade expected. A concrete block s then poured on top of the table with the surface finished as described ve. Provision was made for lifting out these blocks so that they could be laced with others containing desired improvements. In Fig. 4 is shown a tograph of the models under construction.

del Roughness

One of the major problems in the river models was the production of the uired bed roughness to reproduce accurately prototype levels and veloci- s. Artificial roughness beyond that provided by the concrete channel beds s necessary due to the scale distortion of the models. Anticipating this blem, a "Roughness Model" was constructed and tested prior to the com- tion of the river models themselves. In Fig. 6 is shown a photograph of s model which represented, to the same scales and distortion as the river dels, a channel 20,000 feet long, 2,000 feet wide and with a normal depth of feet. These dimensions were chosen as they were of the same order of th and width as the river model channels. In this model various means of ducing roughness were tested, including treatment of the concrete bed it- f, the addition of various thicknesses of expanded metal screening, and the ition of one-half-inch wide vertical metal strips of various heights and at ious spacings. The resulting roughness was measured in terms of the fficient "n" in the Manning formula, and it is felt that a few of the results ained may be of interest. At the normal depth, the mean roughness coef- ents obtained for the various treatments were as follows:

(1) Brushed concrete; n = .015
(2) Single thickness expanded metal screening; n = 0.028
(3) Double thickness expanded metal screening; n = 0.031
(4) Metal strips, 8 inch centres, height = 1/2 depth; n = 0.027
(5) Metal strips, 6 inch centres, height = 1/2 depth; n = 0.033
(6) Metal strips, 4 inch centres, height = 3/4 depth; n = 0.060
(7) Metal strips, 2 inch centres, height = 3/4 depth; n = 0.116

m an analysis of these measurements and others obtained at various depths as concluded that the metal strips would provide the best type of artificial ghness for models 1, 2 and 3. It was found also that the roughness pro- ed by the strips was a function of the strip spacing and the ratio of strip ght to water depth. As it was desired to have an unobstructed water surface



TYPICAL SECTION THROUGH RIVER MODEL SHOWING METHOD OF CONSTRUCTION
FIG. 4

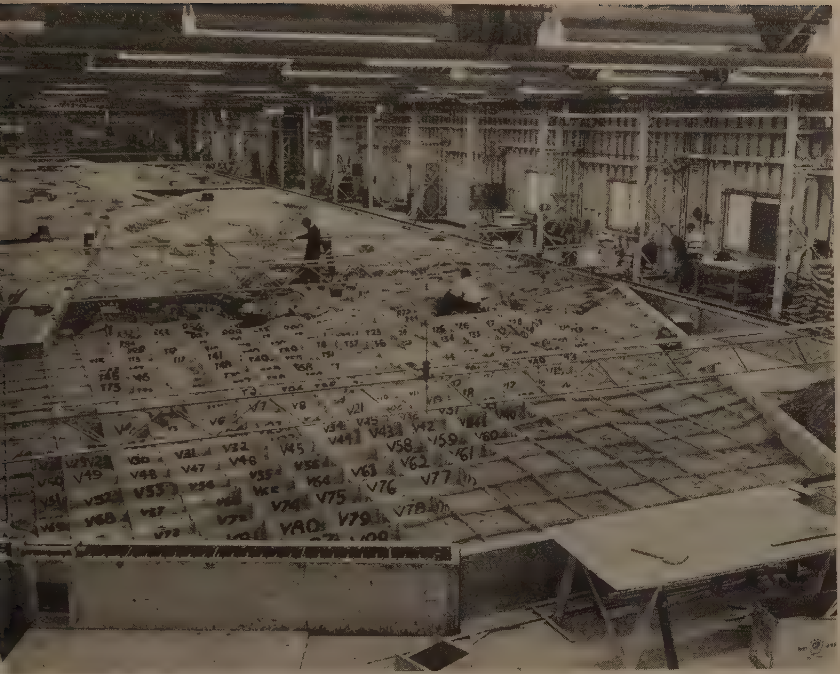


Fig. 5. River Models under Construction

float measurement, it was decided that the basic roughness pattern would produced by one-half-inch wide strips spaced on a six-inch staggered grid, a height above the bed of one-half the depth of mean flow conditions. Actu-y, slots were provided in the model bed on a three-inch staggered grid to ovide flexibility in increasing or decreasing the roughness.

When the models were verified it was found that the basic roughness pattern ıld only be considered as a general guide, as the "form-factor resistance", . that caused by the shape and alignment of the river channel and the esence of islands, constituted an appreciable portion of the total resistance. such points, and where rapids occurred, it was necessary to appreciably ry the strip spacing to provide correct levels. In such cases the strips and slots on the three-inch grid proved to be a very satisfactory and flexible ans of adjusting roughness. For Model No. 9, with less distortion and ger scales, it was found that metal screening over the concrete bed was ficient to produce the desired roughness.

niature Current Meter

In all the river model tests, an accurate measurement of velocities in the del channels was of the utmost importance as the criterion for the channel argement design was the production of a specified mean velocity. Similar- the determination of flow distribution through multiple channels and around ands depended upon the ability to measure velocities. To enable these ve- ities to be measured with sufficient accuracy an improved miniature

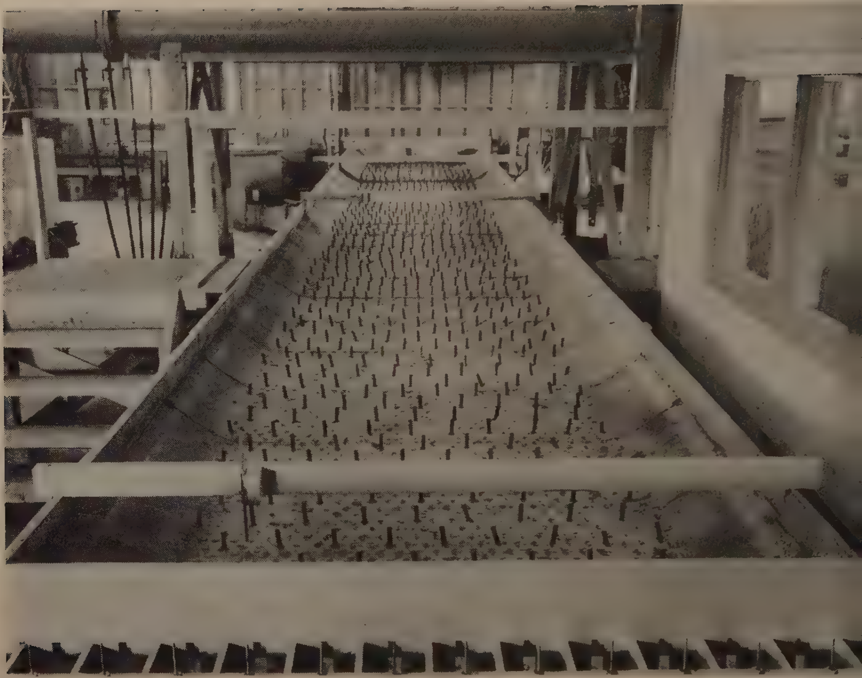


Fig. 6. Roughness Model Showing Metal Strip Test Pattern

propeller-type current meter was developed. The requirements for the met were that it be as small as possible, offer minimum resistance to the water, register accurately model velocities between 0.1 feet per second and 2 feet per second, and be remote indicating. In Fig. 7 is shown a photograph of the current meter developed. Propellers of various pitches and having different numbers of blades enabled velocities as low as 0.07 feet per second and as high as 4.5 feet per second to be measured reliably. A feature of the meter is the remote electronic velocity indicator. Previous electrical indication o velocity required a commutator which inevitably produced a frictional drag. This was eliminated in this meter by devising a means of utilizing changes i electrical resistance of the water itself around the propeller. Basically the instrument consists of an electronic meter which counts the propeller rotations by means of the changes in resistance of the water between the blade and a small electrode, marked X in Fig. 7, located within 0.05 inch of the or of the blade tips. A second electrode, marked Y and remote from the blades was incorporated to balance any changes in water conductivity. An electron circuit counts resistance changes, and the number of changes in a given tim period is a multiple of the rate of rotation of the meter and a function of the water velocity. In operation, the meter is set up in the model channel and a electric cable carried the impulses to the counter which is set up in a convenient location. The operator determines the velocity from the electronic counter and an automatic timing device. A towing channel is provided in the laboratory for periodic calibration of the meters.

Fig. 7. Miniature Current Meter with Electrodes for Frictionless Counting

Measurement of Surface Stream Lines and Velocities

A technique has been developed at Islington for the production of surface stream lines from which surface velocities may be determined. The technique in brief is that a camera is mounted on the laboratory roof and a vertical "aerial" photograph is taken of the model below. Next, in semi-darkness, a number of uniformly-spaced surface floats each containing a lighted candle are released in the model channel at the upstream limit of the photograph. The camera shutter is simultaneously opened and the lighted floats trace surface stream lines on the previously exposed photograph of the channel. An apparatus, attached to the camera, provides a rotating segmented disc which interrupts the field of view once every second. This produces a series of dashes instead of a continuous stream line, the length of the dashes being proportional to the surface velocity. In Fig. 8 may be observed typical results of such a technique.

River Model Accomplishments

The work on the river models extended over a period of several years, and in this paper it is practicable only to record the highlights of the results obtained.

Design of Channel Enlargements

The design of the required channel enlargements in the International Rapids reach was the largest single problem of the model investigation. From the model tests, detailed designs were developed which entailed the removal of some 64,000,000 cubic yards of material by land-based and dredging equipment. The criteria for the design of these enlargements were stipulated in the International Joint Commission's Supplementary Order of Approval for the Project and are as follows:

> "Channel enlargements will be undertaken from above Chimney Point to below Lotus Island, designed to give a maximum mean velocity in any cross-section of the channel which will be used for navigation not exceeding four feet per second at any time, also between Lotus Island and Iroquois Point and from above Point Three Points to below Ogden Island, designed to give a maximum mean velocity in any cross-section not exceeding two and one-quarter feet per second with the flow and at the stage to be permitted on the first of January of any year, under regulation of outflow and levels of Lake Ontario in accordance with Plan of Regulation No. 12-A-9, as prepared by the International Lake Ontario Board of Engineers, dated 5 May, 1955. Downstream from the powerhouses channel enlargements will be carried out for the purpose of reducing the tailwater level at the powerhouses."

In addition to these criteria, navigation interests required a channel having a minimum width of 600 feet with special widening treatment at bends, and with minimum depths of 29.5 feet and 28.5 feet upstream and downstream from Iroquois Dam respectively.

The general nature of the required channel enlargements was first determined by computation, and these preliminary designs were then placed in Model No. 1 and Model No. 2. The initial controlling levels were estimated from backwater computations. These designs were then tested in the models and the velocities compared with the criteria. A very intensive series of tests then were undertaken to refine these designs and test alternatives, to obtain the most economical channels which would satisfy the governing criteria. As the final design was approached, the critical water surface profiles corresponding to the design criteria were also obtained from the model. The design that emerged from these tests is shown in Fig. 1, where hatching indicates the areas of channel enlargement. It is of interest to note that in the Galop Island reach, Model No. 1, improvement of the South Galop channel was found to be of considerable economic value in reducing velocities and excavation in the main Galop enlargement which contained the navigation channel. Similarly at Ogden Island, Model No. 2 proved that economies amounting to many millions of dollars could be effected by diverting more flow to the south of the island by dry excavation, and thus reduce costly and difficult wet excavation in the Plat rapid channel to the north. In the Sparrowhawk Point - Toussaints Island area it was found that the alignment of the river was such

iat excavation somewhat in excess of that required by the criteria was neces-ary to provide satisfactory ice-packing velocities. The navigation channel as located in the model to conform with the requirements, and velocity ieasurements were made throughout to ensure compliance with the criteria. loat pictures indicated the direction of the currents. Comprehensive water urface profile tests indicated the water levels and depths that would occur iroughout the Project. In Model No. 9 a very complete model analysis indi-ated the economic tailrace enlargement, shown also in Fig. 1.

roquois Dam Relocation

In conjunction with an investigation of the foundation conditions for the roquois Dam, a series of tests was run in Model No. 1 to establish the opti-ium location and design of the dam from the hydraulic standpoint. Tests were iade first on the dam at the location and to the design originally shown on the lans approved by the International Joint Commission in its Order of Approval. he dam as shown in these plans was located at Iroquois Point, was curved in lan, and had 40 sluices each 50 feet wide. The tests indicated unsatisfactory onditions, as an abrupt change in direction of the flow occurred at the dam iaking a considerable portion of the sluice area ineffective. After testing six ther plans, it was concluded that much more satisfactory conditions would esult with the dam moved downstream to Point Rockway and with a straight ather than curved alignment. At the new locations, channel enlargements ere necessary upstream to meet the ice-packing velocity requirement. luch-improved hydraulic conditions were found with the dam at the down-tream location and with the associated channel enlargements. In Fig. 8 are hown stream lines with the dam in the original and final locations. The brupt change in direction of the flow at the dam has been eliminated, thus ermitting a reduction in the number of sluices from 40 to 32 while providing n even greater discharge capacity. Many other advantages also resulted; alf the dam could be built in the "dry" at Point Rockway thereby greatly implifying the considerable dewatering problem and the effect on upstream evels at the original location; head losses in the Iroquois reach were sub-tantially reduced resulting in a benefit of many millions of dollars in ultimate ower generation; the foundation conditions were much more favourable at the ew location, which, together with the fewer number of sluices, resulted in a reatly reduced dam cost and an overall saving of close to a million dollars n total construction cost of dam and channel enlargements.

iversion Channels for Long Sault Dam

During the first stage of construction of the Long Sault Dam, it was planned iat the southern portion extending from the United States mainland to Long ault Island would be constructed, and the South Long Sault channel diverted irough "Cut C" shown in Fig. 1. During the second stage, the North Long ault channel would be cofferdammed just upstream from the Long Sault apids, and the flow diverted through "Cut F" diversion channel and through eep diversion sluices left in the first stage of the dam. The design of these iversion channels was carried out in Model No. 3. The criteria for their de-ign was that sufficient discharge capacity must be provided at all times by ie channels so that levels upstream would be satisfactory for the continuance f 14-foot navigation and for the diversion of water through the Massena power anal. In the design of "Cut F" a very thorough study was made to develop its

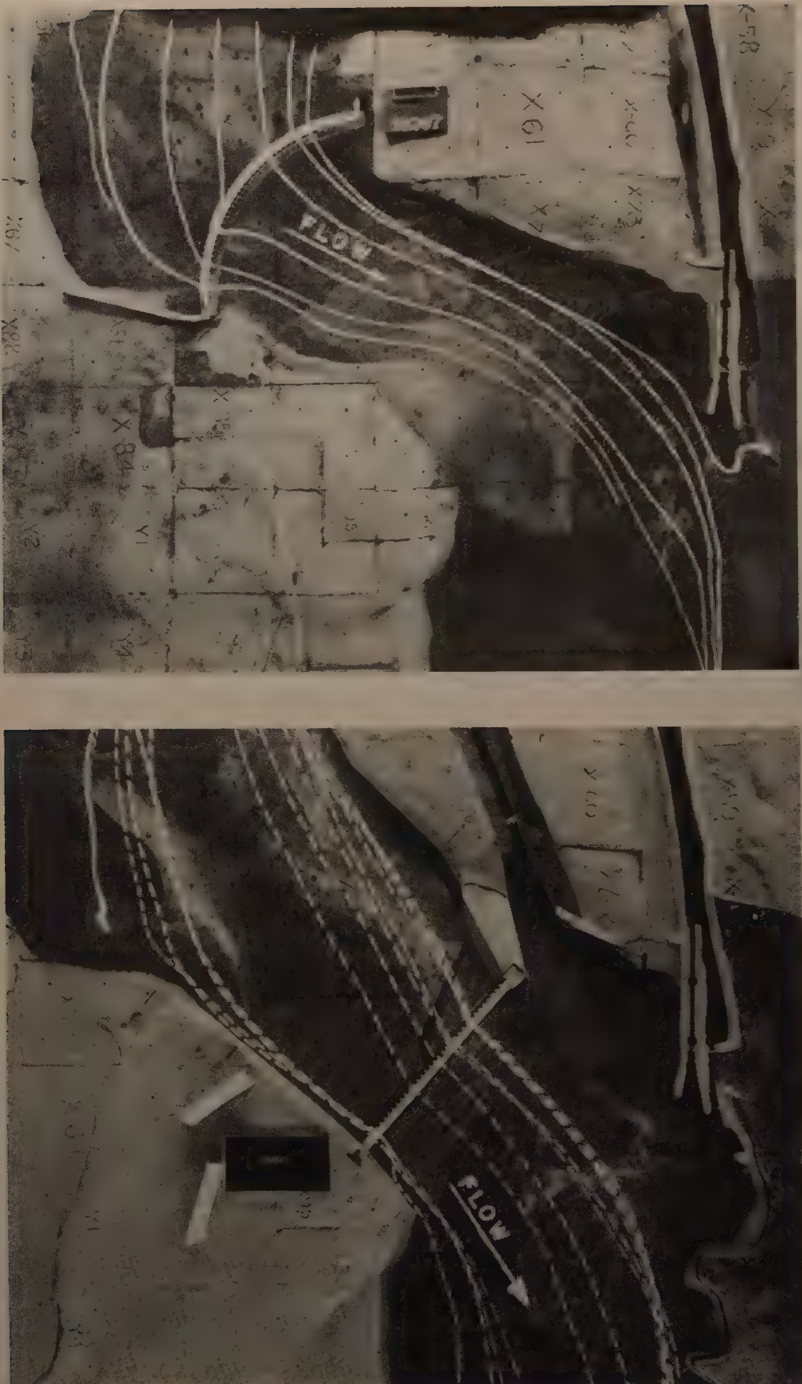


Fig. 8. Surface Stream Lines with Original and Final Locations for Iroquois Dam

ıost economic location and dimensions. This channel, which had to carry lmost the whole St. Lawrence River flow, involved the removal of about ,000,000 cubic yards of material. In Model No. 3 studies were made of the equence and timing of the complete series of construction operations re-uired in the transitions from one stage into another, and a procedure was de-eloped that proved most satisfactory.

iver Control

By the terms of the Order of Approval by the International Joint Com-ıission it was required that the Project be constructed without adversely af-ecting either upstream or downstream interests and that existing 14-foot avigation continue without interruption throughout the construction period. ı effect this required the general maintenance of pre-project levels and out-ows from Lake Ontario and pre-project levels in the International Rapids ection at the locks and in the river used by navigation. These requirements resented a considerable problem in developing an overall plan which would oordinate the construction of the channel enlargements and structures to pro-uce these conditions. In the development of this plan, the three large river ıodels, Nos. 1, 2 and 3, were used. The large natural-scale models of the roquois and Long Sault Dams, Models 8 and 5, were also used in determining ı detail the effects of construction at these dams. Utilizing all these models, step-by-step chronological program of construction was carried out in the ıodels and the effects of the various operations noted. In the earlier stages, ıe timing of various operations was determined such that the effect of one peration would be balanced by the effect of another. In the later stages, the ontrol required at structures was detailed. A coordinated plan was developed o ensure satisfactory level and flow conditions throughout, and this plan roved most satisfactory.

The Long Sault Dam Models

The Long Sault Dam was constructed in three stages. During the first age, that portion containing the southern 13 sluices was built with openings ft generally to river bed elevation in each sluice to pass the full river flow ı the second stage. The permanent sluice gates were adapted to act as di-ersion gates in these openings for control during the second stage. In the econd stage, the remainder of the dam was built, leaving 34 tunnel ports, ach controlled by a gate, to pass the full river flow during the third stage. ı the third stage the ogee sections were completed in the thirteen southern uices. Following this operation, the port gates were closed to raise the ater to the operating level, and the flow controlled by the permanent sluice ates.

In the hydraulic design of the dam, several problems arose which were onsidered to require model investigation; these were:

(1) The need for energy dissipation downstream from the dam during both construction and ultimate operation periods, and operation periods, and if required, the design of an adequate energy dissipator.
(2) The hydraulic loading on all the gates to ensure adequate hoisting capacity.
(3) The discharge capacity of the sluices and ports during the various construction phases.

Description of Models

The Models were built to investigate these various problems. The first, Model No. 10a, was a sectional model of the dam consisting of two piers locat- ed to form a central full sluice with a half sluice on each side. The ogee roll- way was constructed in such a manner that it could be converted to all stages in construction, i.e. diversion sluices, closure ports, or final ogee profile. This model, shown in Fig. 9, was constructed of plexiglass to a natural 1:50 scale and tested in a plexiglass sided flume, 30 inches square and 26 feet long In this model the energy dissipation problem was investigated and the dissi- pator design developed, and exact models of the various gates were made and tested. The results of these gate tests are reported in a later section.

The second model, No. 5, was a comprehensive model of the Long Sault Dam and reproduced, at a natural scale of 1:80, the entire Long Sault Dam, parts of Long Sault and Barnhart Islands, and a portion of the United States mainland. The river channel reproduced extended some 1,000 feet upstream and 2,500 feet downstream from the dam. The model was constructed entirely in concrete and the dam was made with removable sections such that all stages in construction could be reproduced. The energy dissipator developed in the sectional model was also constructed downstream from the dam. A photograph of the model is shown in Fig. 10. The primary purpose of this model was to check the efficacy of the energy dissipator developed in Model 10 over the full range of operating conditions in a comprehensive setting. From

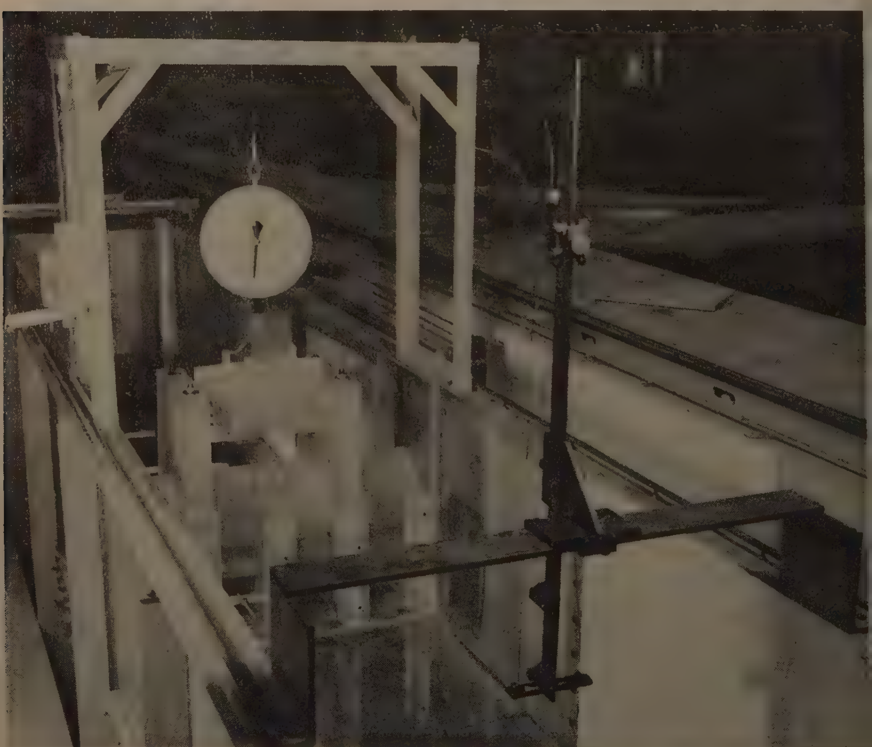


Fig. 9. Sectional Model of Long Sault Dam Showing Gate Load Measuring Equipment

ıst experience, it had been found that in many cases tailwater similarity ınnot be achieved in sectional models due to the restricted channel reproıced. This model was also used to determine the overall discharge ratings the diversion sluices and ports which were affected by high velocity of aproach conditions.

ıergy Dissipating Works

From the model investigation a very severe energy dissipation problem as found to exist at the Long Sault dam both during construction and under nal operating conditions. Because of the nature of the river bed, which opes downward rather steeply downstream from the dam, the tailwater depth as found to be low under all operating conditions, and insufficient to form a ydraulic jump. Preliminary tests indicated that, without energy dissipating orks, the lack of adequate tailwater depths resulted in downstream velocities hich would endanger the safety of the structure even with the relatively low eadwater levels that would exist during the construction periods. The design roblem was unique in that the dissipating works must provide protection for ree entirely different flow conditions which would be encountered during onstruction and under final operation of the dam.

After many trials in the plexiglass-sided flume, where over thirty different esigns were tested, the Recommended Design was developed, and was veried in the comprehensive Model No. 5. It was found that this dissipator maintined a forced hydraulic jump under all flow conditions, and that velocities the order of 75 feet per second with no dissipator were reduced to

Fig. 10. Comprehensive Model of Long Sault Dam

approximately 27 feet per second. In Fig. 11 are shown sketches illustrating the three flow conditions under which the dissipator had to operate.

Although the Recommended Design was the best developed, rock conditions downstream from some of the sluices made it advisable to raise the bottom of the bucket by at least 5 feet. To meet this requirement the Revised Recommended Design was developed, and was shown to be almost as effective as the Recommended Design. Fig. 12 shows details of both energy dissipators.

The Iroquois Dam Models

In the final location selected for the Iroquois Dam, it was possible to build almost half of the dam in the "dry" on the United States shore. During this period, the channel enlargements on that shore were also completed in the "dry", a rim dyke extending a short distance out into the river contained the construction area. When this phase was completed, the rim dyke was progressively removed and the river diverted through the completed portion of the dam. Simultaneously, cofferdams were progressively extended out into the river to dewater the area for the second half of the dam. This work had to be carefully planned to maintain pre-project levels upstream.

It was decided that the following problems during construction and operation required model investigation:

(1) The development of a plan of construction to provide sufficient discharge capacity during each stage of construction. Also required was a knowledge of the velocities and currents that would exist at the cofferdams, particularly during the construction of the second stage cofferdam.

(2) The determination of energy dissipation requirements just downstream from the dam during the later stages of construction, when headwater levels would be raised and tailwater levels would be at or below natural due to the effect of downstream channel enlargements, and the design of necessary energy dissipators.

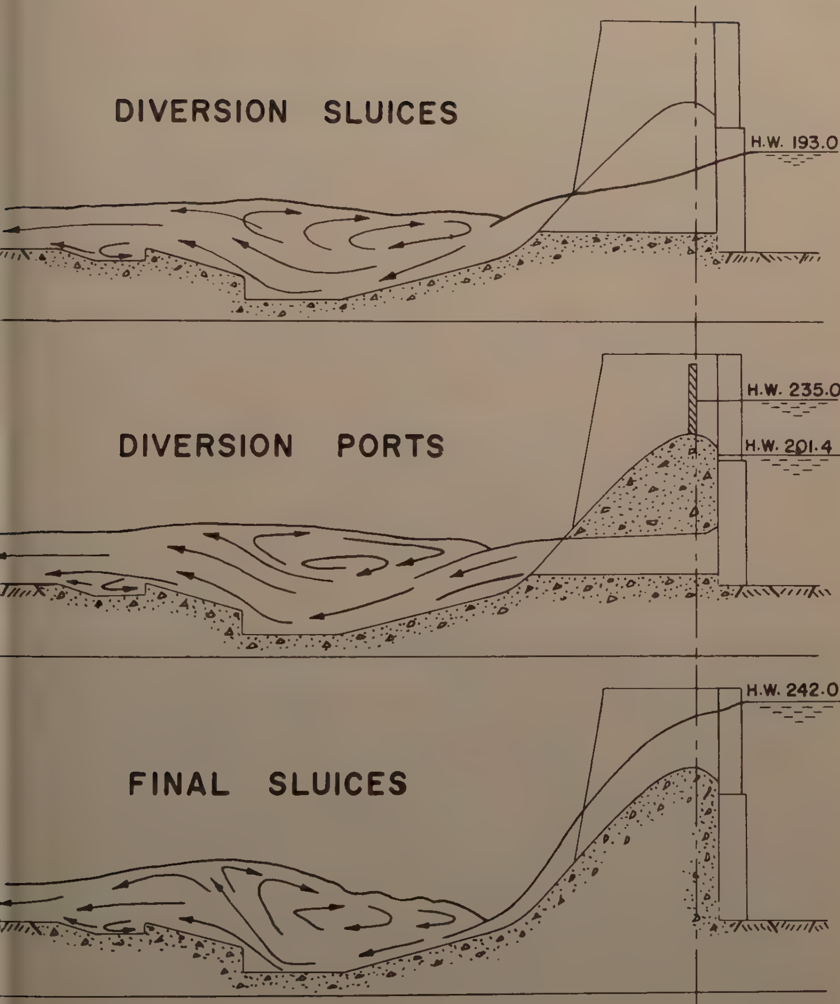
(3) The determination of the hydraulic forces acting on the sluice gates, to assist in selecting the required hoisting capacity.

(4) The development of a discharge calibration of the gates for construction and operation conditions. The pattern of gate openings to provide the most satisfactory river conditions was also desired.

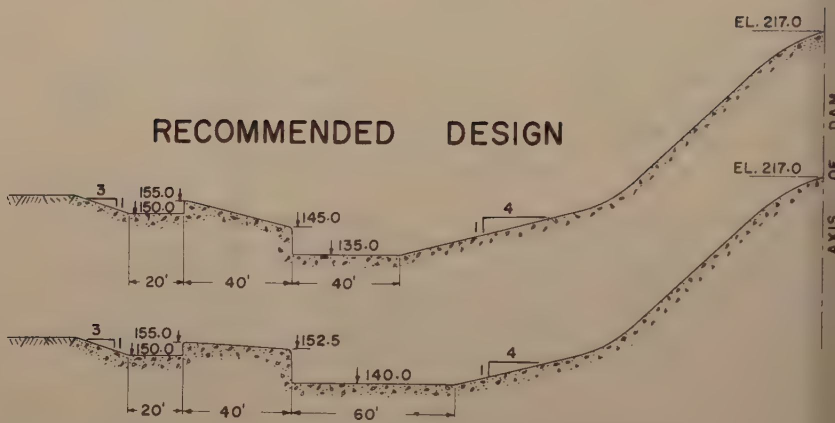
Description of Models

Two models were constructed of the Iroquois Dam to study the foregoing problems. The first was a sectional model constructed of plexiglass to a natural scale of 1:50 consisting of two piers located to form a central full sluice with a half sluice on each side. This model, designated No. 10b, was installed and tested in the testing flume, and was used to investigate energy dissipation requirements and to design the energy dissipator. Exact models of the sluice gates were also built and tested in the model to determine the hydraulic loading on the gates. The results of this gate loading test are reported in a later section.

The second model, Model No. 8, was a large comprehensive 1:80 natural-scale model of the Iroquois Dam, which included the whole dam and its associated channel enlargements, and the downstream approach to the Iroquois Lock. The model reproduced the river upstream for some 5,300 feet and

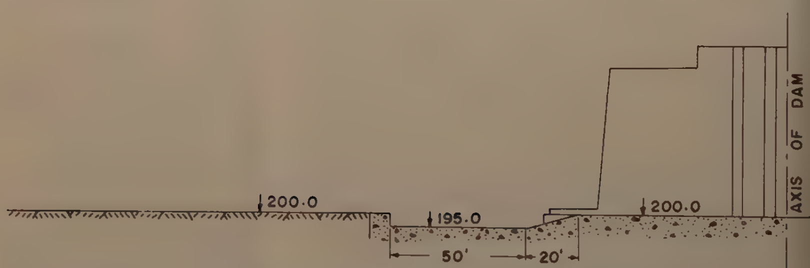


LONG SAULT DAM
THREE STAGES IN ENERGY DISSIPATION
TION OF ENERGY DISSIPATOR - RECOMMENDED DESIGN
FIG. 11



REVISED RECOMMENDED DESIGN

LONG SAULT DAM



REVISED RECOMMENDED DESIGN

IROQUOIS DAM

DETAILS OF ENERGY DISSIPATING WORKS

FIG. 12

vnstream about 3,300 feet. The river portion of the model was constructed irely in concrete, while the dam itself was built of plexiglass. The coffer-n cells for the second stage cofferdam were also built for construction ts. In Fig. 13 is a photograph of the model showing the complete first stage construction and the cofferdams for the second stage.

As in the case of the Long Sault Dam, it was desired to check the per-mance of the energy dissipator in a comprehensive setting. Also, the large del was required to determine in detail the velocities and currents to be ected during construction of the cofferdams during the dewatering of the ond stage area. The discharge calibration of the dam had also to be ob-ned from this model; as under final operating conditions the head across dam is very low and approach conditions affect the results.

ergy Dissipating Works for the Iroquois Dam

Under final operating conditions, the water-level differential across the quois Dam is quite small, and downstream velocities are low. However, ing one period prior to raising the levels in the forebay of the Barnhart and powerhouse, the river was discharged through the sluices of the first f of the dam with headwater levels raised and with tailwater levels at or htly below natural. From model tests of these conditions, downstream ve-ities as high as 45 feet per second were measured, and the need for energy sipation for this period was demonstrated. Several schemes were investi-ed in the flume and the Revised Recommended Design was developed. By action, velocities downstream from the apron of the dam were reduced to roximately 17 feet per second. Additional tests indicated that the pro-ion of ten sluices would be adequate for this period, and the dissipator was

Fig. 13. Comprehensive Model of Iroquois Dam

constructed to protect this number of sluices. Details of the design develop are shown in Fig. 12.

Performance of Dam

From Model No. 8 a complete discharge calibration was obtained for the dam during all stages of construction. This model, however, proved almost indispensable in predicting the optimum gate opening pattern to be used to produce satisfactory conditions downstream. This was particularly so in th period of construction when the headwater level was raised, but tailwater levels were still natural. Under these high head conditions, up to 15 feet, th issuing velocities were necessarily high and unavoidably affected the veloci- ties and currents downstream in the approaches to the Iroquois Lock. From the model, however, a pattern of part-gate openings was devised which pro- duced optimum possible conditions and which enabled navigation to proceed satisfactorily.

The Massena Intake Model

It was planned that diversion of flow through the Massena power canal would be continued during the construction period and also for a limited per after the raising of the St. Lawrence power pool. In the design of the intake it was necessary to ensure that there was sufficient gate capacity at normal river levels, and to determine the performance of the intake structure unde raised water level conditions. It was desired to investigate energy dissipat and to determine the hydraulic loadings on the intake gates under high-head conditions.

Description of Model

To study these hydraulic problems, Model No. 6, a comprehensive 1:60 natural-scale model of the Massena Intake was built. This model reproduc the complete intake itself, the excavated channel in which the intake is locat and a portion of the existing power canal to ensure correct approach and ex conditions. The intake was constructed of plexiglass and the model bed of concrete. Models of the emergency and service gates were also constructe

Results of Model Study

From the model, the best alignment of the structure in relation to the channels was determined to minimize scour in the downstream channel. Limited energy dissipation was found necessary, and this was achieved by ramping up the floor of the sluices near their outlet to direct the jet toward the surface, and directing the jet by an extension of one pier. Interesting r sults were obtained from the gate tests and these are discussed in a later section.

The Powerhouse Models

At the Powerhouse, three models were constructed to investigate the following problems:

(1) The hydraulic loading on the Headgates
(2) The best location for the piezometer taps for the Gibson method of flow measurement
(3) The hydraulic design of the ice sluices and the calibration of the ice sluices control gates.

del of Powerhouse Water Passages

Model No. 7, the Water Passages Model, reproduced the complete power-se water passage from headwater to tailwater. This model, to a 1:36 ural scale, was constructed entirely of plexiglass to enable observations to made of internal flow conditions. Exact models of the headgates were made sheet metal and installed in the model, and were used to determine the tical hydraulic loading on the gates. The results of the headgate tests are en in a later section. Fig. 14 is a photograph of the model which shows the siderable number of Gibson test piezometer connections tested. At this erhouse three large intakes supply each turbine, and the objective of the s was to determine the best cross-sections to be used for the Gibson tests, the best locations for the piezometer connections in each cross-section ecord the true average pressure. The model was found to be most useful his regard.

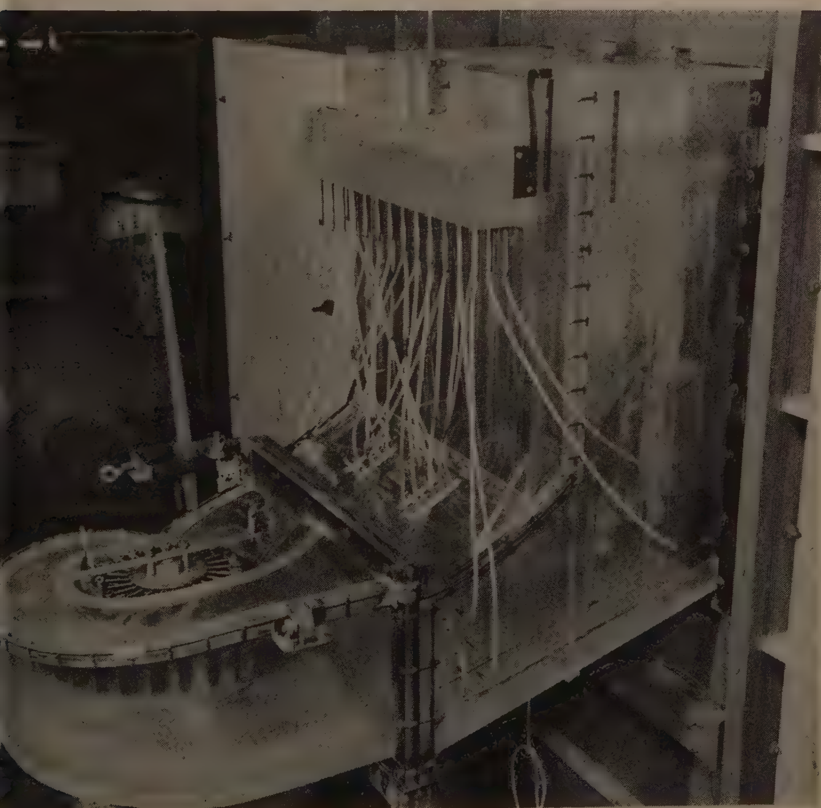


Fig. 14. Powerhouse Water Passages Model

Model of Powerhouse Ice Sluices

At the powerhouse, six ice sluices are provided to enable ice to be discharged downstream. Two adjacent sluices, 75-foot wide, are located at eac end of the powerhouse; while two adjacent sluices, 50-foot wide, are located at the middle of the plant. Each sluice is controlled by a drum-type gate. When operating, these sluices discharge ice and water through the powerhou from headwater to tailwater in a closed water passage which does not run fu To study the behaviour of ice being discharged through these sluices, and to develop a design which would operate safely, Model No. 4 was constructed o plexiglass to a natural scale of 1:36. Paraffin wax was used to represent ic and Fig. 15 shows a photograph of the model in action. Earlier designs tend to trap the ice inside the passage at the foot of the slope with the ice churnin heavily against the structure. However, by adjusting the sluice profile and selecting the right floor and ceiling elevation in the lower portion, it was found possible under all conditions to discharge the ice and water at high velocity through the passage and into the tailwater without the hydraulic jump entering the passage.

The general requirements for the drum-type gates controlling the ice sluice discharge were determined in Model No. 4 and further model tests were made by the gate manufacturer. To provide a discharge calibration fo

Fig. 15. Powerhouse Ice Sluice Model

gates, Model No. 11 was constructed of the upper portions of the ice chute a natural scale of 1:36. Two 75-foot wide sluices and two 50-foot wide ices were reproduced and the discharge calibration obtained.

Hydraulic Loading on Gates

In a number of the model studies described earlier it was mentioned that ts were run to determine the vertical hydraulic loading on the water-trolling gates of the various structures. It is considered desirable to re-t on all these results in this section, as it is of interest to compare the sults obtained. In all seven different project gates were tested to determine vertical hydraulic loads which could be expected under the most severe erating conditions. Four of the gates were to be completely submerged in eration while the remainder were not. The submerged gates were the verhouse headgate, the Long Sault Dam tunnel port gate, and the Massena ake service and emergency gates. Unsubmerged gates tested, were for the quois Dam and for the diversion sluices and spillways of the Long Sault n. Because of the interesting nature of the results, a summary of the test dings has been included in this paper. It should be noted that in all the test ults the effects of buoyancy, gate weight and friction in the checks have n eliminated, and the loads shown are therefore due to vertical hydraulic ion only. The apparatus used in the gate tests is shown in Fig. 9, where gate is suspended from a hoist through a spring balance used to measure vertical loading. Friction between the gate and the checks is eliminated pulling the gate upstream just free of the checks. This is accomplished by pecial harness which assures a horizontal pull and also that the gate is ting just free of the checks.

ts on Tunnel Port Gates at Long Sault Dam

Perhaps the most interesting tests were those on the temporary gates, d to close the tunnel ports in the Long Sault Dam. While these gates were smallest of those tested, their weight being about 25 tons each, it was nd that they were subject to the most severe loading conditions. As shown Fig. 16, investigations on the preliminary design indicated that a maximum l of 154 tons could be expected if the holes required for eventual concreting ne ports were plugged, and 124 tons if the holes were left open. By alter-the shape of the bottom section to 45° and then to 60° as shown in Fig. 16, as possible to reduce the maximum downpull to 73 tons and 54 tons re-ctively.

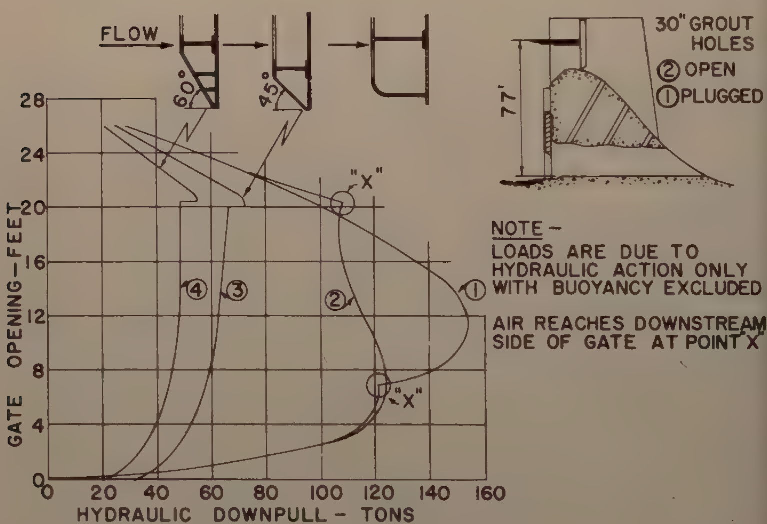
ts on Other Project Gates

The tests on the other project gates produced a variety of results which shown in Fig. 17, where the grouping depends upon whether the gates were merged or not. While it is not practicable to analyse the results in detail, ain observations may be of interest.

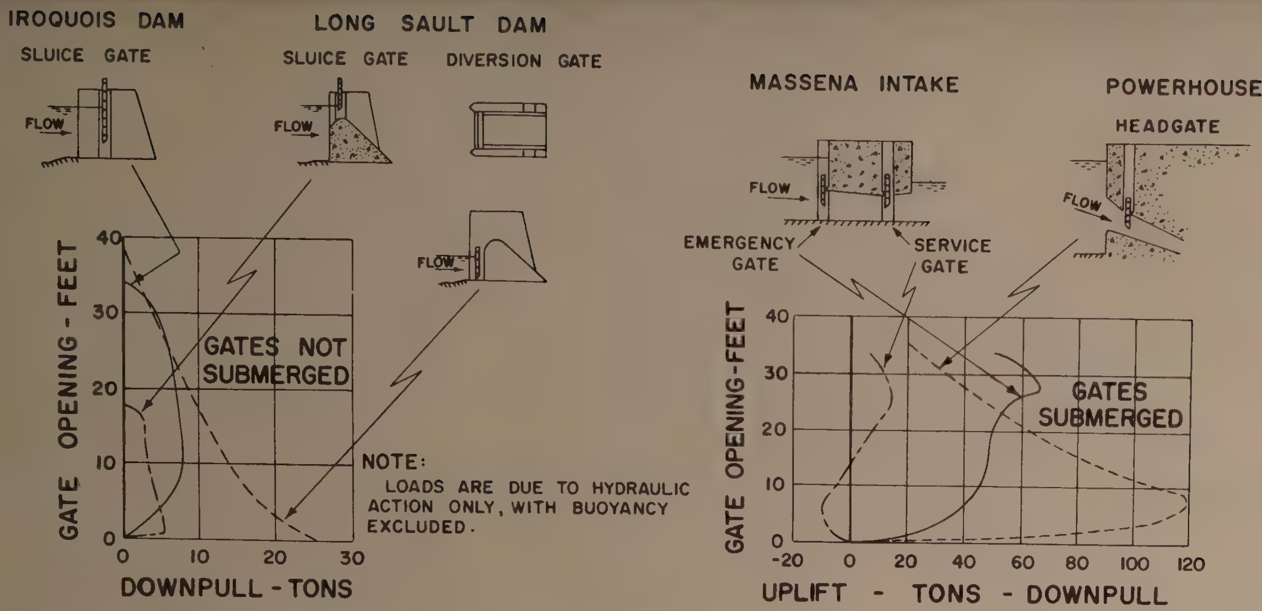
For unsubmerged gates conventionally installed with upstream skin plates, as at the Iroquois Dam and Long Sault Dam spillways, it would appear neither downpull or uplift is a significant force. The long Sault Dam di-sion sluices, being an adaption of the final sluice gates, were not con-ional in their installation because the sluice width was only 80 per cent

that of the gate, with the result that water collected on the unsheeted downstream members and thus increased the indicated downpull.

The three submerged gates were generally similar in size and illustrate the effect of some of the varying factors. At Massena the skin plate was on the downstream side of the emergency gate and on the upstream side of the service gate. The large hydraulic downpull on the emergency gate, and the tendency for uplift to occur on the service gate, indicates that the position o the skin plate influences greatly the hydraulic loading. These loads may hav a considerable bearing on the required hoisting capacity or the ballast required to ensure that the gate will close. The powerhouse headgate was test under turbine runaway conditions with the other two gates in the supply passages closed, and the large resulting downpull shows the effect of the increase in head across the gate.



MEASURED VERTICAL HYDRAULIC FORCES O DIVERSION PORT GATES AT LONG SAULT DA FOR ORIGINAL AND REVISED DESIGNS
FIG. 16



RESULTS OF TESTS TO DETERMINE VERTICAL HYDRAULIC LOADS ON GATES AT POWERHOUSE, MASSENA INTAKE, IROQUOIS DAM AND LONG SAULT DAM

FIG. 17

CONCLUSIONS

From the foregoing review, it is apparent that a very considerable effort has been made by the Power Authority of the State of New York and the Hydro-Electric Power Commission of Ontario to test thoroughly all aspects in the hydraulic design of the St. Lawrence Power Project. It is considered that the models have proven to be indispensable in providing designs which carry the maximum assurance of successful performance and at the same time make possible maximum economies. In the case of this Project, economies were effected which exceeded by many times the cost of the models. Information from the models enabled each step in construction to be taken with confidence, in the knowledge that its effect had been tested in advance and found to be within the criteria. Another considerable advantage of the models, was the ability to demonstrate in advance proposed plans and procedures to supervisory boards and other interested agencies. In all, it is felt that the models have played an essential part in the Project.

ACKNOWLEDGEMENTS

It is apparent that the successful carrying out of a comprehensive model testing program for a project such as the St. Lawrence requires the full co-operation of many organizations and individuals and is in essence a team effort. In this connection mention should be made of the full support and co-operation received from the field and office engineering organizations of the Power Authority of the State of New York, the Hydro-Electric Power Commission of Ontario, and Uhl, Hall and Rich, engineers for the Authority.

With regard to the Model investigation itself, special mention must be mad of Mr. D. G. Harkness, who supervised the program, and the able and resourceful team of Ontario Hydro engineers who carried out the work in the laboratory.

Journal of the

HYDRAULICS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

MECHANICAL ANALOGS AID GRAPHICAL FLOOD ROUTING[a]

Closure by Max A. Kohler

MAX A. KOHLER,[1] M. ASCE.—The author is indeed flattered by Mr. Clark's discussion of his paper. The analogies drawn by Mr. Clark, as for example between the instantaneous hydrograph and the template routing, are realistic and accordingly lead to a better understanding of flood-wave phenomena. His caution that lag is in reality translatory storage is well taken.

a. Proc. Paper 1585, April, 1958, by Max A. Kohler.
1. Chf. Research Hydrologist, U. S. Weather Bureau, Washington, D. C.

NORTHEASTERN FLOODS OF 1955: RAINFALL AND RUNOFF[a]

Closure by Tate Dalrymple

TATE DALRYMPLE,[1] A. M. ASCE.—The author wishes to thank Mr. urner for his careful review of the paper. The discussion presents conuiderable material that adds to the subject of the paper, and thereby improves .

The author would hesitate to assign a frequency to a given flood that neessitated a long extrapolation of knowledge. Generally, a 50 year recurrence terval is about as far as is justified, except in a few instances where long rm knowledge is available, as in New England. A project now being cometed by the Geological Survey has uncovered considerable historical flood formation in New England, and promises to form a basis for extending flood equency curves to 300 years. An extrapolation beyond this point seems unise, regardless of the need for evaluating greater floods.

Proc. Paper 1662, June, 1958, by Tate Dalrymple.
Chf., Floods Section, Surface Water Branch, Water Resources Div., U. S. Geological Survey, Washington, D. C.

WAVE FORCES ON SUBMERGED STRUCTURES[a]

Discussions by I. Alterman and N. Hamlin

I. ALTERMAN.[1]—This paper is a welcome addition to the vast but attered literature on the subject, as evidenced by the large number of Refer-ces following the article.

The addition of semi-empirical data of horizontal forces on barge models a timely and very important contribution to problems arising in the ever-reasing number of sea-structures executed. The article is partly of inter-for those who prefer a theoretical approach and partly for those who prefer ractical one.

For compensation, it might be of interest that the writer has made use of theoretical formulae, such as (6), (7) of the article, or rather an inte-ation of those, and the results have shown a dramatical application:

A series of 320 steel piles, 16" diameter, driven 33 feet into the sea-bed e served as a temporary jetty. Once the superstructures had to be dis-ntled, the writer calculated the maximum waves which these piles could hstand freely, as winter storms approached.

The computation showed that 16-ft. waves, with a period of 15 seconds ld cause breaking stresses to the piles. Orders were given accordingly cut-off the piles as quickly as feasible, before the stormy period would roach.

Before all piles could be cut off, a first storm came, with waves 12 feet h and 15 seconds period striking the piles. An inspection of the piles wed no detrimental effect. The Resident-Engineer thus questioned the uracy of the formulae and slowed down the work. A check on the formulae this case proved that these waves could not cause over-stressing of the es, because the bending-moments were proportional to the square of wave ghts.

The next storm actually brought 15 feet waves, while only 4 out of the 320 es were left. All four piles were bent at the sea-bed level beyond repair.

The above full scale experiment may add evidence to the applicability of theory.

On the other hand, it would be desirable to complete the article by intro-ing data relating to horizontal cylindrical submerged elements.

Examples of such elements are:

a) Horizontal connections of pile bend.
b) Submarine pipes.

---

a. Proc. Paper 1893, November, 1958, by Ernest F. Brater, John S. McNown and Leslie D. Stair.

1. Head, Div. of Concrete Pipes, Water Planning for Israel Ltd. (TAHAL), Tel Aviv, Israel.

(c) Floating submarine vessels.
(d) Horizontal elements of “Jumbo” structures etc.

A useful formula for the forces exerted by waves on an immersed cylinde may be found in literature.[1,2]

Thus, the resistance to be taken up by the cylinder is:

$$R = 4\pi^2 g^3 \rho r^4 c^{-4} e^{-2gf/c^2} \qquad \text{(1}$$

where:

g = acceleration due to gravity

ρ = sea-water density

r = radius of cylinder

c = wave celerity

f = depth of centre of cylinder below mean water level.

It is assumed that $f = r$, i.e. the cylinder is not too close to the free water level.

In the special but common case that the cylinder is at the sea-bed[3] leve (e.g. a pipe), the above formula becomes very simple:

$$R = \frac{4\pi^2 g \rho r^4}{f^2 e^2} \qquad \text{(2}$$

while

$$c = \sqrt{g f} \qquad \text{(3}$$

Or, if we denote the constant

$$A = \frac{4\pi^2 g \rho}{e^2} \qquad \text{(4}$$

we simply get:

$$R = A \frac{r^4}{f^2} \qquad \text{(5}$$

This simple formula shows the main difference between the action of the waves on a plate or a vertical cylinder to a horizontal cylinder. In the latte case the wave action is shown to increase with the 4th power of the radius ar inversely with the square of the depth. This explains why a small diameter pipe may be laid without any support on the sea-bottom, while a larger one will suffer more than an equivalent vertical pile.[4] Also a submarine vesse will “bounce” in shallow water while in deep water, since the action decreas with the square of the depth, it will be quite steady.

Also, a flat plate will be affected by forces equivalent to the exposed area (see e.g. formula (8) of the article), while a submerged horizontal cylinder may have forces proportional to a higher degree, e.g. the 4th power.

REFERENCES

1. H. Lamb, Hydrodynamics; Cambridge University-Press, 1932.

I. Alterman, Submarine Outfall Pipe; Technion, Israel Institute of Technology, Haifa, 1958.

I. Alterman, Pipes Within Water; Symposium on Water Conduits; Ministry of Agriculture, Water Planning for Israel Ltd., Mekoroth Water Co. Ltd., Association of Engineers and Architects in Israel. Second edition 1956.

I. Alterman, A Test on a Sea-Outfall Pile Bent; Journal of Association of Engineers & Architects in Israel, Vol. XIV, No. 4 (Nov. - Dec. 1956).

N. HAMLIN.[1]—The authors' tests are a welcome addition to the store of wledge available to the designer of offshore structures. To the writer's wledge these tests were the first to measure wave forces methodically on arate fixed bodies representative of submerged barges in the absence of porting piles.

The significant conclusions from the tests and the authors' analysis thereof first, the predominant wave forces on the barge-like models were inertial second, values of mass coefficient needed to determine these forces from retical calculations compared favorably with values estimated from purely retical considerations or determined experimentally by oscillating models till water.

n order to estimate wave forces on a rectangular barge, the writer's ctice has been to use a mass coefficient based upon data reported by Yu in authors' Reference 19. An overall comparison of wave force data from el tests of a complete drill rig, together with preliminary data from the ors' tests which were made available to the writer's company as one of sponsoring organizations at the time of the tests, indicated that the Yu nula (modified to make it dimensionless) brought about reasonable corre-on of the measured forces. However, mass coefficients from the authors' iminary measurements of horizontal forces on the barge models were ificantly lower than those from the Yu formula. The comparison referred ; reported in the writer's discussion of a 1957 paper before the A.M.E.[2] The final horizontal barge forces reported in the present paper considerably higher than the preliminary data indicated and lead to mass ficients which now agree favorably with the Yu formula. (The final values about 15% higher than formula values.) This tends further to confirm the ata as providing means for estimating mass coefficients for wave forces ubmerged rectangular barges with acceptable engineering accuracy.

owever, the results of tests with a flat plate are surprising in that the ıal mass derived from the tests was high compared to theory. In addition, e appeared to be a substantial drag force acting on the plate which could be explained by a drag coefficient higher than has been found from tests eady flow. Regarding the flat plate tests, a possible source of scatter in lata and of high transient forces might be the tendency of shed vortices e vicinity of the model to retain their strength and thereby to continue to ence the forces developed on the plate. This would seem to be more of a effect rather than inertial. Have the authors any evidence in support of an explanation? In this connection, it is of interest that the flat plate

---

ethlehem Steel Co., Shipbuilding Div., Central Technical Dept., Quincy, lass.

echtin, Steele and Scales, P. 633-681, 1957 S.N.A.M.E. Transactions.

test data reported by Yu tends to confirm the theoretical value of $C_m V$ for a infinitely long plate which, as the authors state, is equal to the volume of a circumscribing cylinder. Incidentally, with regard to the authors' reservations concerning C_m values obtained from high frequency oscillations, the Yu tests in water are reported for frequencies ranging from about .29 to .11 c.p.s., which are lower than the wave frequencies used in the authors' tests.

There are presently two interpretations of Eqs. (4) and (5) in use. Many who have worked with wave theory, including the authors, take the symbol (d + z) to mean the instantaneous particle height above the bottom. The writ is of the opinion that (d + z) should mean the average particle height, in othe words, the height of the center of the particle's orbit. The latter interpreta enables one to depict orbital motion and other features of the wave in a mor realistic manner. An apparent confusion is indicated, but is of little consequence in analysing the authors' tests, inasmuch as both interpretations lea to equal but opposite upstream and downstream horizontal inertial forces. However, assuming (d + z) to represent the average particle height, theoretical vertical inertial forces are slightly larger when directed upward than wl downward for bodies fixed in the wave but near the surface. The tests with the barge models appear to confirm this tendency, but also show an unexpec trend for upward forces to exceed downward when the models were near the bottom.

The authors are to be congratulated for a clear presentation of test resul which are of fundamental importance.

SNOWMELT RUNOFF[a]

Discussion by H. C. Riggs

ɪ. C. RIGGS,[1] A. M. ASCE.—The authors propose a method of computing rate of runoff from melting snow on the premise that, for practical use, off from melt must be related to generally available meteorological data. nputing snowmelt runoff on a daily basis requires first a relation between actual melt water produced on a unit area and the causative factors. The relation probably would be one derived from data obtained on a small on which melt water and the meteorological factors were accurately sured. The second part of the problem is that of routing the melt water stream channel and thence downstream to the gage site.

'he subject paper treats these two parts as one. A disadvantage of this roach is that the reliabilities of the several procedures (transferrence of eorological data, distribution of runoff from one day's melt to streamflow several days, altering of discharge data on the beginning day of melt to ct a trial-and-error solution) cannot be evaluated separately.

'he authors' results are incomplete for the purpose of verifying their pro- ıres. Stream discharge is not related to melt potential alone; the amount vailable water in the snow pack and the precipitation during the melting od must be included in order to define a usable relation. The wide di- ence of the curves of the authors' Fig. 8 indicates the need for additional ables. The equation

$$Y = 27.4\ X - 1.88\ X^2$$

ely puts the divergent curves of Fig. 8 into register at a melt potential of nches. This equation was an acceptable fit in only 7 of the 15 years tried therefore, does not constitute verification of the method. No equation of type, however reliable, could be used for forecasting because results are n, not in quantitative forms, but as a percentage of cumulative runoff at nches melt potential. This cumulative runoff at 7.0 inches melt potential ıknown at the time of forecast.

he most reliable data available should be used in developing and testing a method. It is unfortunate that the streamflow records in the selected n are not of highest accuracy. However, the use of stream discharges re- d by the authors does not lend confidence to their results. The "ice index" od is not new. It is a simplification of a graphic method described by (1) in 1913. Present practice is described by Moore.(2)

---

roc. Paper 1834, November, 1958, by J. Harold Zoller and Arno T. Lenz.

ydr. Engr., U. S. Geological Survey, Washington 25, D. C.

A method of forecasting the rate of runoff from snowmelt is of practical use only when the causative factors can be predicted in advance or when the is appreciable lag between occurrence of the causative factors and the resu ing runoff. Compliance with one of these necessary conditions needs to be demonstrated.

REFERENCES

1. U. S. Geological Survey Water-Supply Paper 337.
2. Measuring Streamflow under Ice Conditions, ASCE Proc. Paper 1163, February 1957.

PROCEEDINGS PAPERS

ne technical papers published in the past year are identified by number below. Technical-division sorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air sport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hy-ics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil anics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors , divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with me 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate rs in the Journals, the symbols after the paper number are followed by a numeral designating the issue particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) h indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division g 1958.

VOLUME 84 (1958)

1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 T3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 39(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 M2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 56(ST3)[c], 1657(SM2)[c].

: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 L2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 O3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 Y3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 01(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 T4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 M3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

JST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 O4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 O4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

EMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 58(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 67(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775 R3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784 R3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

BER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 98(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806 Y5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815 T6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824 M4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c]

MBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 41(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 50(SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858 U3)[c].

MBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 T8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 W5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 85(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

VOLUME 85 (1959)

ARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 00(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 09(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 18(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926 L1), 1927(HW1), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

RUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 41(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 50(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958 O1), 1959(CO1).

CH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968 A2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977 W1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 86(IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

L: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998 M2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007 W2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022 Y5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 31(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 40(AT2)[c], 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

scussion of several papers, grouped by divisions.